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Geotechnical & Environmental Services

PRELIMINARY GEOTECHNICAL SITE INVESTIGATION

40 WOLFES ROAD - NEIKA

PROPOSED EXTENSION

Client: Hamish and Rachel Ostler

Certificate of Title: 161408/3

Investigation Date: 23/03/2026

Refer to this Report As

Enviro-Tech Consultants Pty. Ltd. 2026. Preliminary Geotechnical Site Investigation Report for a Proposed Extension, 40 Wolfes Road - Neika. Unpublished report for Hamish and Rachel Ostler by Enviro-Tech Consultants Pty. Ltd., 8/05/2026.

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Limitations of this report

In some cases, variations in actual Site conditions may exist between subsurface investigation boreholes. This report only applies to the tested parts of the Site at the Site of testing, and if not specifically stated otherwise, results should not be interpreted beyond the tested areas.

The Site investigation is based on the observed and tested soil conditions relevant to the inspection date and provided design plans (building footprints presented in Attachment A). Any site works which has been conducted which is not in line with the Site plans will not be assessed. Subsurface conditions may change laterally and vertically between test Sites, so discrepancies may occur between what is described in the reports and what is exposed by subsequent excavations. No responsibility is therefore accepted for any difference in what is reported, and actual Site and soil conditions for parts of the investigation Site which were not assessed at the time of inspection.

This report has been prepared based on provided plans detailed herein. Should there be any significant changes to these plans, then this report should not be used without further consultation which may include drilling new investigation holes to cover the revised building footprint. This report should not be applied to any project other than indicated herein.

No responsibility is accepted for subsequent works carried out which deviate from the Site plans provided or activities onsite or through climate variability including but not limited to placement of fill, uncontrolled earthworks, altered drainage conditions or changes in groundwater levels.

Footing exposure classification is presented on a layer-by-layer basis. In practice, some layers may be removed during excavation or replaced as part of site cuts and fills, while others may be incorporated within the building envelope. The information should therefore be regarded as guidance only, and the designer must assess the actual founding conditions and make the final determination of concrete strength, curing and cover requirements.

At the time of construction, if conditions exist which differ from those described in this report, it is recommended that the base of all footing excavations be inspected to ensure that the founding medium meets that requirement referenced herein or stipulated by an engineer before any footings are poured.

The preliminary slope stability assessment presented herein is based on inferred parameters derived from field observations and DCP correlations only. No laboratory shear strength testing has been completed at this stage. The assessment should therefore be regarded as preliminary and subject to revision following additional investigation and testing.

Executive Summary

Envirotech completed a preliminary geotechnical site investigation for the proposed extension at 40 Wolfes Road, Neika. The investigation included test pit excavation, Dynamic Cone Penetrometer (DCP) testing, soil logging, laboratory testing, preliminary footing assessment, and a preliminary slope stability assessment.

The Site is located on steep south-facing terrain and comprises variable clay-dominant soils with locally gravelly and sandy horizons. The investigation identified high plasticity clay, shallow low-strength soils, variable fill conditions, elevated moisture conditions, and evidence of poorly managed surface water around the existing and proposed building footprint. The Site has therefore been classified as a Class P Site in accordance with AS2870 due to the identified problematic ground conditions.

A preliminary slope stability assessment was undertaken using an interpreted engineering ground model derived from DCP resistance trends, field observations and inferred engineering parameters. The preliminary model returned a calculated factor of safety of 1.28, below the adopted target criterion of 1.50 for long-term stability. This indicates that the Site may be vulnerable to shallow slope instability under the currently adopted conditions and assumptions.

The current shallow soil moisture conditions are considered to be significantly influencing the geotechnical assessment outcomes at the Site, particularly the reduced shallow soil shear strength conditions identified near TP04. These moisture conditions are interpreted to reflect a combination of uncontrolled runoff from paved areas, poorly managed stormwater, wastewater disposal downgradient of the building, ponding around near-level benches, and localised water use during excavation around tree roots. These factors appear to be contributing to wet, low-strength clay conditions within the upper soil profile.

Although elevated moisture conditions were also encountered at depth, potentially associated with groundwater discharge from a nearby geological fault, the shallow moisture conditions are considered most influential to the interpreted shallow failure mechanism and preliminary slope stability outcomes identified during this assessment.

In Envirotech’s opinion, improved drainage and surface water management may significantly improve the geotechnical outlook for the proposed development by reducing shallow soil moisture conditions and increasing near-surface soil strength over time. However, engineering judgement is required to determine whether drainage improvement alone will be sufficient to adequately manage the identified slope stability risks, or whether additional measures such as further investigation, remove-and-replace earthworks, retaining systems, suspended footing systems, or other engineered solutions are required.

The Site also exhibits significant reactive clay and tree-removal effects associated with a previously removed *Eucalyptus regnans* adjacent to the proposed building area. The resulting tree-affected characteristic surface movement is estimated to be approximately 100 mm, consistent with tree-affected Class E footing conditions.

Given the identified geotechnical constraints, the proposed development should proceed only with detailed structural and geotechnical engineering input. Water management should form a critical component of the final design. In particular, all surface water runoff encroaching upon or residing within the Site should be intercepted and diverted away from the existing and proposed building footprint to reduce infiltration into the moisture-sensitive clay profile.

Site Investigation

The Site investigation is summarised in Table 1.

Table 1 Summary of Site Investigation

Client	**Hamish and Rachel Ostler**
Project Address	40 Wolfes Road - Neika
Council	Kingborough
Planning Scheme	Interim Planning Scheme
Inundation, Erosion or Landslip Overlays	None
Planning Trigger (Land Stability)	A land stability assessment is required due to proposed earthworks exceeding 1 m depth in accordance with Clause 14.4.3 of the Kingborough Interim Planning Scheme.
Proposed	Extension
Investigation	Fieldwork was carried out by an Engineering Geologist on the 23/3/2026
Site Topography	The Site is defined by the proposed building and works area which has a very strong slope of approximately 31% (17°) to the south
Site Drainage	The Site receives overland flow runoff directly from the north.
Soil Profiling	Four investigation holes were excavated from surface level around the proposed extension (Appendix A):
Investigation Depths	The target excavation depth was estimated at 2.3 m. Test pit TP01 was sampled to 3.3 m, test pit TP02 was sampled to 2.7 m, PT03 was tested to 2.3m and test pit TP04 was sampled to 3 m. Borehole logs and photos are presented in Appendix B & C.
Soil moisture and groundwater	All recovered soil at the site ranged from moist to wet. Seepage and groundwater conditions were encountered at shallow depth, with wet soil conditions observed throughout much of the investigated profile.
Geology	According to 1:25,000 Mineral Resources Tasmania geological mapping (accessed through The LIST), the geology comprises of: Permian Unit Puo, contact metamorphosed by Jurassic dolerite.
Wind Loading	
Terrain category:	TC2
Shielding Classification:	NS
Topographic Classification:	T0
Wind Classification:	N2

Introduction and Scope of Works

This report presents the preliminary geotechnical site investigation completed for the proposed extension at 40 Wolfes Road, Neika. The investigation was undertaken to assess subsurface conditions relevant to foundation design, Site classification, soil reactivity, drainage, and preliminary landslip susceptibility.

The Site is not located within a mapped landslip hazard overlay. However, the proposed development includes earthworks exceeding 1 m in depth, which triggers a requirement for a land stability assessment under the Kingborough Interim Planning Scheme.

Field observations and preliminary interpretation indicate that the slope contains variable clay-dominant materials, locally gravelly, with groundwater or seepage encountered at shallow depth. These conditions are relevant to both foundation design and slope stability.

This report should therefore be read as a preliminary geotechnical site investigation. It documents the investigation completed to date and identifies the need for further targeted assessment before a final Landslide Hazard Assessment can be completed.

Purpose of this Report

- The purpose of this report is to:
- document the Site conditions and subsurface profile encountered during investigation
- provide preliminary Site classification and footing design information
- summarise the interpreted ground model based on test pits, DCP testing, logging and laboratory testing
- identify preliminary landslip-related concerns requiring further assessment
- recommend additional investigation and testing required to refine slope stability assessment

Status of Landslip Assessment

A preliminary slope stability assessment has been undertaken to assess the potential impact of the proposed earthworks on slope stability. The assessment indicates that slope stability may be sensitive to the adopted shear strength parameters and groundwater conditions. As the current assessment is based on inferred parameters derived from DCP correlations and field observations, the results are considered indicative only. Further targeted investigation is required to obtain representative strength parameters before finalising the land stability assessment.

The preliminary findings presented in this report indicate that the Site may require a combined geotechnical and structural engineering solution rather than a conventional residential footing system.

Site Classification Summary

In accordance with AS2870 – 2011 and after thorough consideration of the known details pertaining to the proposed building and associated works (hereafter referred to as the Site), the geology, soil conditions, soil properties, and drainage characteristics of the Site have been classified as follows:

CLASS P based on the following problematic ground conditions identified at the site:

- Fill other than SAND encountered/proposed at the Site at a thickness greater than 0.4 m (up to approximately 0.5 m).
- Based on DCP testing, soft soil was encountered at the site at depths of up to 0.6m at the base of the fill in PT03.
- Low bearing capacity soil was encountered with allowable bearing capacities of less than 100 kPa to a depth of up to 0.3 m in TP02; 0.7 m in PT03; 0.9 m in TP04.
- Variable high plasticity clay
- Substantial proposed cut and fill within the structural footprint
- Tree-removal effects
- Groundwater and poor drainage conditions
- Preliminary slope stability concerns identified during assessment.

Notwithstanding the problematic ground conditions identified at the Site, the calculated characteristic surface movement corresponds approximately to Class H2 reactivity under natural conditions and tree-affected Class E conditions following consideration of the removed *Eucalyptus regnans* adjacent to the proposed development area.

Geological Setting

The Site is mapped within Permian Parmeener Supergroup sedimentary rocks of the Malbina Formation / contact-metamorphosed equivalent. Jurassic dolerite is mapped upslope of the Site, with a mapped fault/contact between the dolerite and underlying sedimentary units (Figure 1). This geological setting may be relevant to the observed elevated moisture conditions, as groundwater may preferentially migrate along geological contacts, fault structures, and permeability contrasts between dolerite, talus and sedimentary units.

Findings

Subsurface Conditions

The subsurface profile encountered across the investigated area generally comprised clay-dominant soils with variable sand and gravel content. The logged soil units were variable and gradational, and the boundaries between materials were not generally observed to represent sharply defined or laterally continuous stratigraphic layers. For this reason, the detailed soil profile should be interpreted as a record of local material variation rather than a series of discrete continuous layers.

In TP01 (see Figure 2), higher gravel content soils were encountered in the upper profile to approximately 1.5 m depth. Below this depth, the profile transitioned into high plasticity CLAY, typically described as CLAY with sand and trace gravel. This clay-dominant profile continued to the depth of investigation, with no clear transition to a substantially stronger basal unit encountered within TP01.

In TP04, the upper profile also comprised clay-dominant soils, including Sandy CLAY and CLAY with sand and trace gravel. A relatively discrete high plasticity CLAY interval was encountered between approximately 1.6 m and 2.1 m depth, broadly consistent with the clay-dominant interval observed in TP01. Below this, CLAY with sand and trace gravel continued to approximately 2.9 m depth, where a cobble-rich Silty Gravelly CLAY unit was encountered. This cobble-rich material was not identified in the other investigation locations and appears to represent a localised stronger or coarser deposit.

Overall, the Site is interpreted to comprise a variable clay-dominant slope profile, locally gravelly and sandy, with high plasticity clay forming the main material relevant to foundation and slope stability assessment. The variability in gravel and sand content complicates direct correlation between test locations; however, the dominant engineering behaviour is interpreted to be controlled by the clay matrix and its moisture-dependent strength.

In summary, TP01 shows gravelly soils in the upper profile and clay-dominant material below, while TP04 records clay-dominant material to about 2.9 m before the cobble-rich Silty Gravelly CLAY unit.

Geotechnical Ground Model

The detailed logged soil profile has been simplified into engineering units to better represent the behaviour of the slope materials for geotechnical assessment. This simplification is necessary because the logged soil boundaries are variable and gradational, and do not appear to form laterally continuous stratigraphic layers across the Site.

The engineering ground model has been interpreted primarily from DCP resistance trends, field consistency, moisture condition, and the relative position of TP01 and TP04 within the proposed building area. The model is therefore behaviour-based, rather than a literal layer-by-layer stratigraphic model.

For the purpose of preliminary slope stability assessment, the profile has been divided into the following engineering units:

- Firm High Plasticity CLAY
- Stiff High Plasticity CLAY
- Very Stiff Medium Plasticity CLAY

The Firm High Plasticity CLAY is interpreted to extend to approximately 2.1 m depth in TP01 and approximately 1.6 m depth in TP04. This unit is considered the most relevant to shallow slope stability because it includes the lower-strength clay materials identified from DCP testing.

The Stiff High Plasticity CLAY was encountered below the firm clay profile and appears to continue to depth in TP01, with no clear basal transition encountered within the depth of investigation. In TP04, a stronger transition was interpreted from approximately 2.6 m depth, below which substantially higher resistance material was encountered.

The Very Stiff Medium Plasticity CLAY is interpreted as the lower-strengthened engineering unit within the simplified slope model. This unit was most clearly indicated in TP04, where stronger material was encountered below approximately 2.6 m depth. The continuity of this unit across the slope is inferred and should be treated as interpretive.

The engineering unit boundaries shown on the cross-section are inferred and should not be regarded as sharp geological contacts. They represent interpreted transitions in material behaviour for the purpose of preliminary stability modelling and further investigation planning.

Preliminary Landslip Considerations

The Site is not located within a mapped landslip hazard overlay. However, a land stability assessment has been triggered because the proposed development includes earthworks exceeding 1 m depth under Clause 14.4.3 of the Kingborough Interim Planning Scheme.

Several landslide features have been mapped by Mineral Resources Tasmania within approximately 1 km of the Site (Figure 1). Of these, MRT landslide point 8577 is considered the most relevant comparator, as it is located west of the Site in a broadly similar topographic position downslope of the same mapped fault structure.

It is noted, however, that landslide 8577 is mapped within Quaternary colluvial / talus material, whereas the Site is mapped within Permian sedimentary rocks of the Malbina Formation / contact-metamorphosed equivalent. This represents a significant difference in mapped geological setting and means that landslide 8577 should be treated as a contextual indicator of regional slope susceptibility rather than direct evidence of instability at the Site.

The Site is located on a strong south-facing slope, with overland flow received from upslope areas to the north. The investigation encountered a variable clay-dominant soil profile, with perched groundwater or seepage observed at shallow depth as well as deep soil moisture potentially originating from a deeper regional groundwater flow associated with upgradient faulting. These conditions are relevant to slope stability, particularly where proposed excavation, filling, retaining or drainage works may alter the existing slope geometry or groundwater regime.

The model indicates a potential shallow slide scenario with a calculated factor of safety below the adopted screening target. This result suggests that slope stability is sensitive to the adopted shear strength parameters and groundwater conditions.

The preliminary model does not constitute a final slope stability assessment. The current assessment relies on inferred parameters only, with no laboratory shear strength testing completed at this stage. Further targeted sampling and laboratory testing are required to confirm representative strength parameters, refine the slope stability model, and complete the final slope stability assessment.

Recommendations

General

Due to the identified problematic ground conditions and preliminary land stability concerns, the footing, retaining and drainage systems should be designed by suitably qualified engineers experienced in reactive clay and sloping site development.

Land Stability and Earthworks

The preliminary slope stability assessment indicates that the stability of the Site is sensitive to the adopted soil strength parameters and groundwater conditions. Based on the inferred parameters, the calculated factor of safety does not meet the adopted screening criterion for long-term stability.

The current assessment has been undertaken using DCP-derived correlations and field observations, which provide indicative estimates of soil strength only. These methods do not directly measure the effective stress parameters required for slope stability analysis. Given the sensitivity of the model to these parameters, further investigation or mitigation is required to confirm acceptable stability.

At this stage, three potential pathways are available to address the identified land stability risk:

Further Investigation

Targeted undisturbed sampling (e.g. Shelby tube or block sampling) and laboratory shear strength testing (triaxial testing) may be undertaken to determine representative strength parameters for the clay profile. These results would be used to refine the slope stability model and confirm whether the Site can achieve acceptable stability under current conditions.

Remove and Replace

The preliminary slope stability model indicates a potential shallow failure mechanism with a calculated factor of safety of 1.28, below the adopted target criterion of 1.50. The modelled shallow slip zone includes fill to approximately 0.5 m depth overlying wet, moisture-sensitive high plasticity clay.

Given these conditions, the project engineer should consider whether removal and replacement of unsuitable materials within the structural footprint is required. This may include excavation of fill and weak or moisture-sensitive clay, replacement with engineered fill of suitable strength and drainage characteristics, and incorporation of appropriate subsoil drainage.

Any remove-and-replace option should be designed and certified by the project engineer and should demonstrate that the completed earthworks provide acceptable foundation performance and land stability.

Drainage and Groundwater Management

Surface water and subsurface water management are considered critical to the long-term performance and stability of the proposed development.

Observations made during the investigation indicate that drainage around the existing and proposed building footprint is poorly managed. Surface water appears to be entering the proposed development area from multiple sources, including upslope runoff, uncontrolled runoff from paved areas, local stormwater discharge, wastewater disposal downgradient of the building, and water used during excavation around tree roots. This water is not being effectively collected or diverted away from the building area and is contributing to wet, low-strength soil conditions within the clay profile.

The current shallow soil moisture conditions are considered likely to reflect a combination of uncontrolled runoff from paved areas, poorly managed stormwater, wastewater disposal downgradient of the building, ponding around near-level benching, and recent use of water or pressure washing to excavate soil around tree roots.

Immediate drainage rectification is recommended to improve Site soil conditions and create a more favourable geotechnical setting for the proposed development. This should include interception of upslope runoff, collection and controlled discharge of runoff from paved areas, positive surface grading away from the building footprint, prevention of ponding on benches, and controlled discharge of stormwater well away from the slope-sensitive area.

Wastewater disposal immediately downgradient of the proposed building should be avoided unless specifically assessed and approved by the project geotechnical and hydraulic engineers. Additional subsurface moisture loading downslope of the structure may adversely affect soil strength, slope stability and long-term footing performance within the clay profile.

Drainage improvement may improve shallow soil strength conditions over time; however, it should not automatically be relied upon as a standalone stability solution. The project engineer should assess whether drainage rectification alone is sufficient to manage the identified slope stability risks, or whether further geotechnical assessment, testing, remove-and-replace works, retaining design, or other mitigation is required.

Foundation and Structural Design

The proposed development includes substantial cut and fill within the structural footprint and is located within an area of variable high plasticity clay affected by groundwater and previous tree removal. Based on the findings of the preliminary slope stability assessment, reliance on shallow moisture-sensitive clay soils within the building footprint is not recommended without further detailed geotechnical verification.

The suitability of a conventional shallow slab-on-ground footing system must be assessed and confirmed by the structural engineer, taking into account the problematic ground conditions, tree-removal effects, proposed cut and fill, drainage conditions and preliminary land stability considerations identified in this report.

The footing and retaining system should be designed by a suitably qualified structural engineer experienced in reactive clay and sloping sites. Deep bored piers founded below the weaker clay profile may be considered by the structural engineer in preference to shallow founded systems within the cut/fill zone.

A suspended floor system does not remove the requirement for detailed assessment of retaining, drainage and global slope stability.

The final footing and retaining design must be confirmed by the structural engineer. Envirotech recommends that the design avoid reliance on uncontrolled fill, shallow high plasticity clay, or moisture-sensitive soils within the proposed structural footprint.

Dispersive soil Management

Findings

The results presented in Appendix E indicate:

- Soil tested from the Site is either not dispersive (Class 4 or greater) or only slightly dispersive (Class 3).

Site specific recommendations

- No specific dispersive soil management measures are considered necessary based on the Emerson Class testing completed to date.
- The soils at the Site are only slightly dispersive with a low risk of soil/tunnel erosion.

Soil Exposure Classification

The soil has been tested for salinity impacts on footings in accordance with AS2870, as well as preliminary pH testing as a proxy to potential sulphate aggressivity.

- With an exposure classification of A2, at a minimum, 25 MPa concrete is generally recommended with 40 mm cover using a damp-proofing membrane or 50mm cover without. A minimum curing time of 3 days is recommended.
- It is also essential to refer to other applicable construction standards to ensure compliance with all relevant guidelines and best practices.

Plumbing

Refer to hydraulic design drawings for detailed plumbing advice and requirements.

Refer to Table 2 to assess soil movement (Ys) around pipework for different depth ranges.

*Table 2 Millimetres soil movement (Ys) for determining plumbing requirements for various soil depths **

Building	Profiles	P*	E Ys >75	H2 Ys 60-75	H1 Ys 40-60	M Ys 20-40	S Ys 0-20	A Ys 0
Dwelling	All	Yes	0-0.1	0.1-0.5	0.5-1.1	1.1-1.8	1.8-3	>3

* Depths in this table are based on surfaces at the time of testing and do not allow for the influence of any additional fill added to the soil profile unless the Iss calculation depth has been modified based on the proposed cut and fill (see 'Footing Minimum Target Depths'). Where additional fill is proposed (and not indicated in the attached plans) Enviro-Tech are to be advised of final FFL's so the Site classification can be recalculated according to the specific fill reactivity and thickness used in the design.

Class H1, H2 and E

When pipework service trench basses fall within Class H1, H2 and E depth range as shown in Table 5, and all plumbing recommendations herein have been implemented, the drainage system must be designed in accordance with AS2870 which includes but are not limited to ensuring:

- Avoid penetrating edge beams of rafts and perimeter strip footings when possible. If necessary, detail them to allow movement. Use closed-cell polyethylene lagging around all stormwater and sanitary plumbing drainpipe penetrations through footings. The lagging should be at least 20mm thick on Class H1 sites and 40mm thick on Class H2 and Class E sites. Vertical penetrations do not need lagging. Note: Sleeves that allow equivalent movement may be used instead of lagging.
- The drains attached to or emerging from underneath the building shall incorporate flexible joints immediately outside the footing and beginning within 1 m of the building perimeter to accommodate a total range of differential movement in any direction equal to the estimated characteristic surface movement of the site (ys). In the absence of specific design guidance, the fittings or other devices that are provided to allow for movement shall be set at the midposition of their range of possible movement at the time of installation, so as to allow for movement equal to 0.5ys in any direction from the initial setting. For example, in the case of H1 there needs to be an allowance for 30mm of flex in any direction of the initial central setting point.
- Drainage under a slab shall be avoided where practicable.

NOTES:

- Pipes may be encased in concrete or in recesses in the slab when provided with flexible joints at the exterior of the slab.
- The methods used should comply with the AS/NZS 3500 series.
- For drains constructed in unstable grounds (Site classifications other than Class A and Class S), AS3500.2:2021 Appendix G of AS3500.2:2021 should be referred for general advice.
- Hot and cold water pipes should not be installed under slabs, and shall be installed in accordance with AS/NZS 3500 series.
- Performance solutions are required for Class E and P Sites in which case a structural engineer and hydraulic designers' input is required for an agreed solution.
- Refer to Hydraulic consultant for advice and design of all hydraulic services required to be documented with the above requirements prior to building and plumbing approval.

Site Drainage and Groundwater Management

Surface water and subsurface water management are considered critical to the long-term performance and stability of the proposed development.

The Site currently exhibits poor drainage conditions, including water ponding and infiltration within the proposed work area. Investigation identified wet high plasticity clay extending to depth, including moisture contents at 39% within TP01 at approximately 3.2 m depth. These findings indicate that elevated moisture conditions are not limited to near-surface soils and may extend deeper into the clay profile.

Upslope runoff should be intercepted and diverted away from the proposed building envelope using cutoff drains or equivalent drainage measures. Surface water should not be permitted to accumulate within the proposed cut/fill benches or adjacent to retaining structures and footings.

Positive surface grading should be established immediately following earthworks, with all stormwater directed to a lawful discharge point located away from the slope-sensitive area.

Wastewater disposal immediately downgradient of the proposed building should be avoided unless specifically assessed and approved by the project geotechnical and hydraulic engineers. Additional subsurface moisture loading downslope of the structure may adversely affect soil strength, slope stability and long-term footing performance within the clay profile. Wastewater disposal should therefore be relocated or specifically assessed by the project engineer and hydraulic designer to ensure that groundwater conditions are not adversely impacted.

Drainage improvement alone should not be relied upon as a standalone stability solution unless subsequent testing confirms that soil strength conditions have improved sufficiently.

All test pit locations and the former tree excavation/backfill area should be reinstated with suitable compacted material to minimise future settlement and prevent preferential infiltration of surface water into the clay profile.

Where stormwater flow paths, drains, spoon drains or surface runoff are proposed to pass over or near backfilled test pits or the former tree excavation, the drainage pathway should be lined or otherwise sealed to prevent water entering the backfilled zones. This is particularly important where backfilled excavations occur upslope of, adjacent to, or within the proposed building footprint.

Backfilled excavations should not be allowed to act as preferential seepage pathways into the moisture-sensitive clay profile.

Temporary Site Drainage

It is recommended that drainage protection works (cut off drains/mounds) are put in place above (upgradient of) the work area to prevent water and sediment from accumulating in and around footings and reduce infiltration, erosion, softening of the clay profile and potential instability associated with saturated ground conditions.

Trees and Tree Removal Effects

The calculated characteristic surface movement for the Site, excluding tree effects, is Ys = 74.5 mm, rounded to 75 mm, corresponding to Class H2 reactivity.

A large *Eucalyptus regnans*, estimated to have been approximately 40 m high, was formerly located approximately 3 m from the proposed building envelope. In accordance with Appendix H of AS 2870, the influence distance for a single tree may be taken as 1.0HT, giving an estimated influence distance of approximately 40 m. The ratio Dt/HT is approximately 0.075, which is less than 0.5; therefore, the full tree-induced movement has been adopted for assessment purposes.

For the adopted design suction depth of Hs = 3.0 m, Appendix H gives a maximum design tree drying depth of approximately 3.4 m and a maximum additional suction change of 0.38 pF for a single tree. Application of the Appendix H tree suction profile to the Site soil reactivity profile gives an estimated additional tree-related movement of approximately 25 mm.

The resulting tree-affected characteristic surface movement is therefore estimated to be approximately 100 mm, which exceeds the normal H2 range and is considered consistent with tree-affected Class E footing conditions.

As the tree has been removed, the relevant design case is considered to be moisture recovery and potential clay heave following tree removal, rather than ongoing tree drying. In accordance with Appendix H of AS 2870, the calculated footing design mound height for the tree-removal case is approximately 75–80 mm.

In addition to reactive soil movement, retained root systems and future root decay may locally disturb and soften the soil profile beneath or adjacent to the proposed development area.

Given the proposed earthworks, proximity of the former tree location to the structural footprint, and the identified moisture-sensitive clay profile, the footing and retaining system should be specifically designed by the structural engineer for tree-affected reactive clay conditions and land stability considerations identified within this report.

Important Engineering Requirement

The preliminary findings presented in this report do not constitute approval of the proposed footing, retaining, drainage or wastewater design. The final structural, retaining, drainage and footing systems must be designed and certified by suitably qualified engineers taking into account the geotechnical constraints identified within this report.

Foundation Maintenance

Details on appropriate site and foundation maintenance practises from the CSIRO BTF 18 Foundation Maintenance and Footing Performance: A Homeowner's Guide are presented in Appendix G of this report along with Australian Geoguide (LR8) Hillside Construction Practice (Appendix H).

K Taylor

Kris Taylor, BSc (hons)

Environmental & Engineering Geologist

Notes About Your Assessment

The Site classification provided and footing recommendations including foundation depths are assessed based on the subsurface profile conditions present at the time of fieldwork and may vary according to any subsequent *Site works* carried out. *Site works* may include changes to the existing soil profile by cutting more than 0.5 m and filling more than 0.4 to 0.8 m depending on the type of material and the design of the footing. All footings must be founded through fill *other than* sand not exceeding 0.4 m depth or sand not exceeding 0.8 m depth, or otherwise a Class P applies (AS2870 Clauses 2.5.2 and 2.5.3).

For reference, borehole investigation depths relative to natural soil surface levels are stated in borehole logs where applicable.

In some cases, variations in actual Site conditions may exist between subsurface investigation boreholes. At the time of construction, if conditions exist which differ from those described in this report, it is recommended that the base of all footing excavations be inspected to ensure that the founding medium meets the requirement referenced herein or stipulated by an engineer before any footings are poured.

The site classification assumes that the performance requirements as set out in Appendix B of AS 2870 are acceptable and that site foundation maintenance is carried out to avoid extreme wetting and drying.

It is the responsibility of the homeowner to ensure that the soil conditions are maintained and that abnormal moisture conditions do not develop around the building. The following are examples of poor practises that can result in abnormal soil conditions:

- The effect of trees being too close to a footing.
- Excessive or irregular watering of gardens adjacent to the building.
- Failure to maintain Site drainage.
- Failure to repair plumbing leaks.
- Loss of vegetation near the building.

The pages that make up the last six pages of this report are an integral part of this report. The notes contain advice and recommendations for all stakeholders in this project (i.e. the structural engineer, builder, owner, and future owners) and should be read and followed by all concerned.

References

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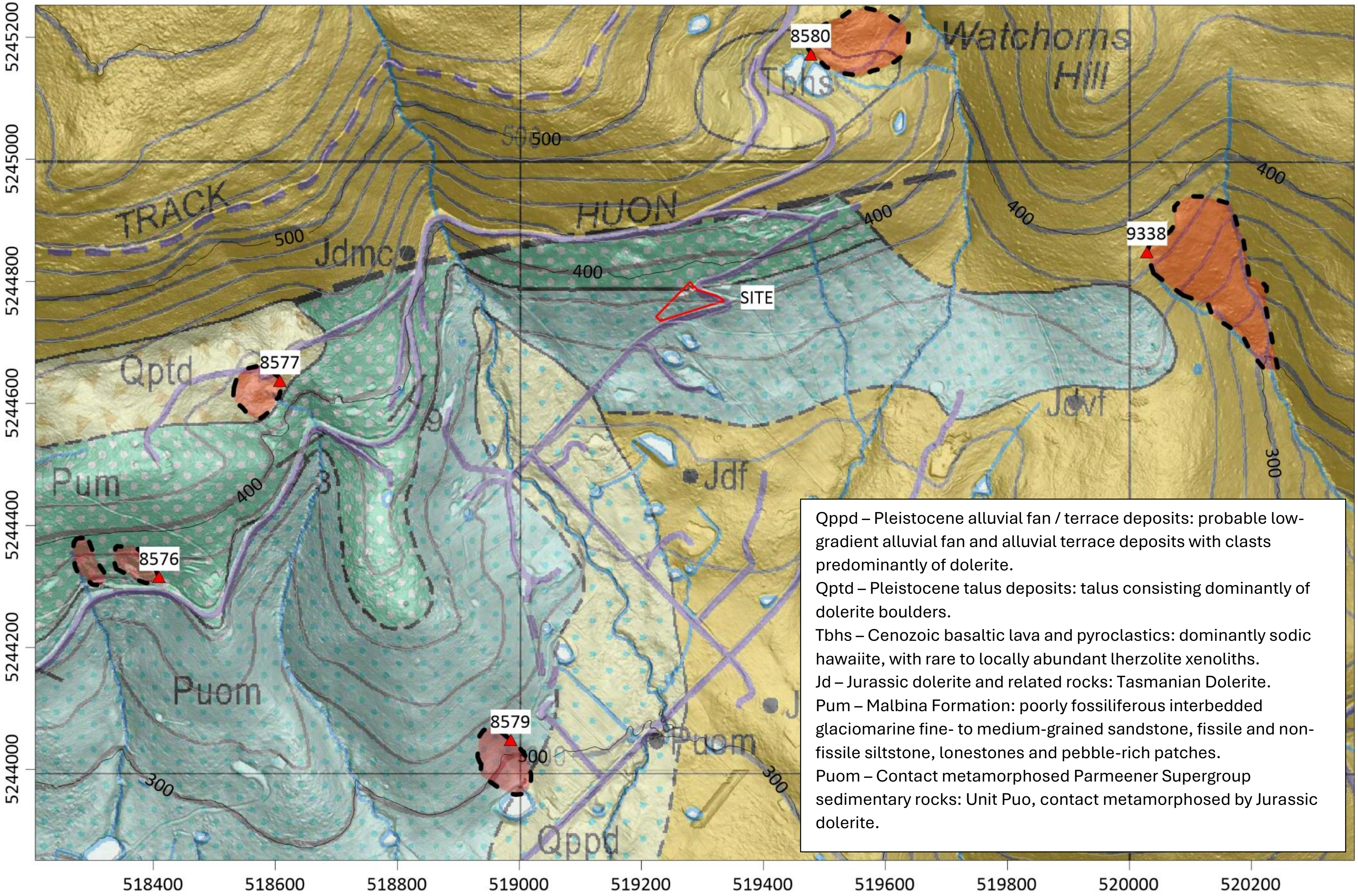
Appendix A Mapping



Qppd – Pleistocene alluvial fan / terrace deposits: probable low-gradient alluvial fan and alluvial terrace deposits with clasts predominantly of dolerite.

Qptd – Pleistocene talus deposits: talus consisting dominantly of dolerite boulders.

Tbhs – Cenozoic basaltic lava and pyroclastics: dominantly sodic hawaiite, with rare to locally abundant lherzolite xenoliths.

Jd – Jurassic dolerite and related rocks: Tasmanian Dolerite.

Pum – Malbina Formation: poorly fossiliferous interbedded glaciomarine fine- to medium-grained sandstone, fissile and non-fissile siltstone, lonestones and pebble-rich patches.

Puom – Contact metamorphosed Parmeener Supergroup sedimentary rocks: Unit Puo, contact metamorphosed by Jurassic dolerite.

Figure 1 Geological mapping sourced from Mineral Resources Tasmania.   MRT mapped landslip points and polygons.

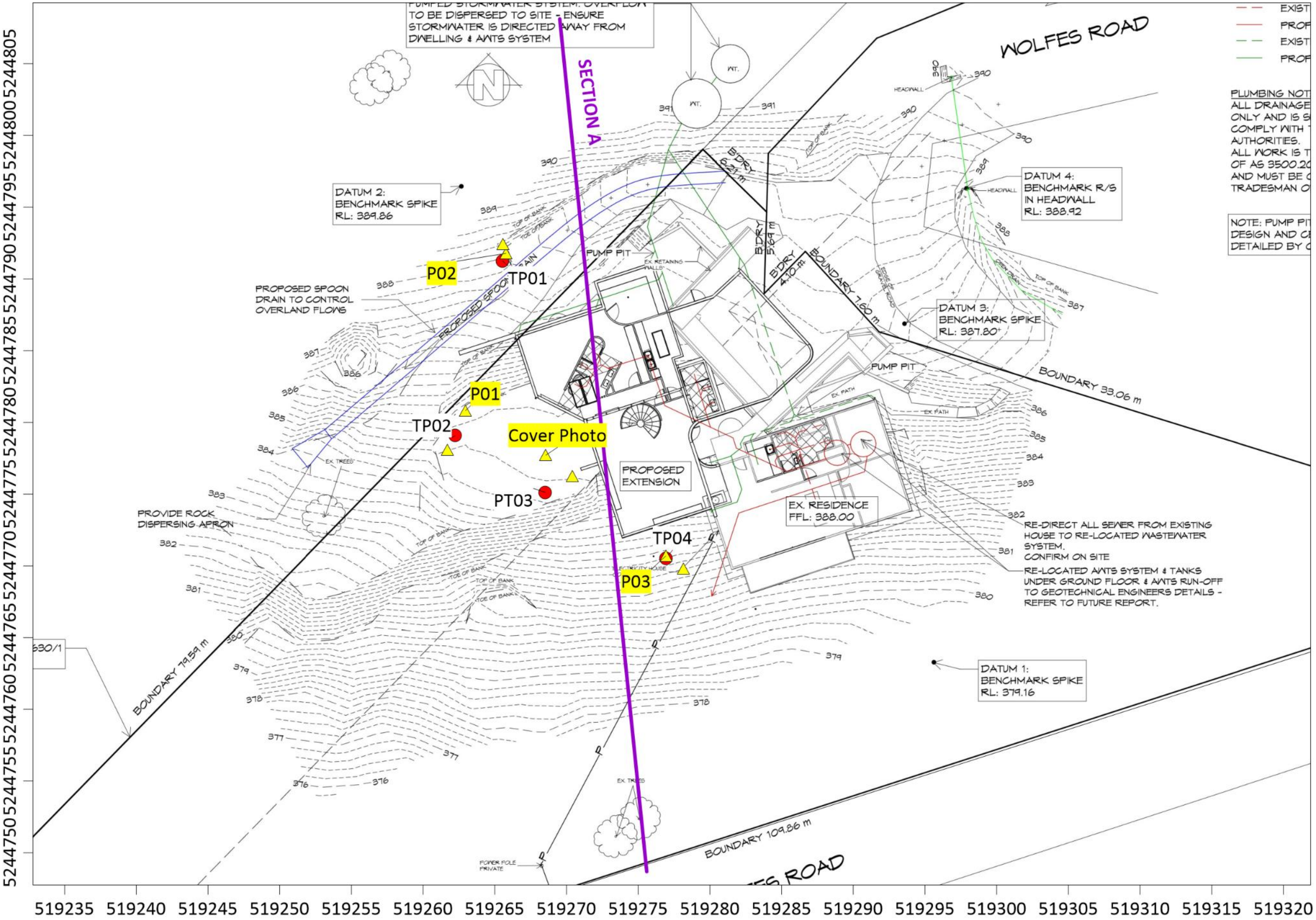


Figure 2 Test Pit and Photograph Locations. Cross Section location.

Appendix B Borehole Logs

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Positioning: GDA94 & mAHD

ASSESSMENT: Geotechnical Site Investigation
STRUCTURE: Extension
EASTING: 519265.5
NORTHING: 5244791.3
ACCURACY
HORIZ: 0.1m **VERT:** 0.1m

Test Pit : TP01

DATE TESTED: 23/03/2026
LOGGED BY: Y. Karki
ELEVATION: 387.3

LOCATION: 40 Wolfes Road - Neika
CLIENT: Hamish and Rachel Ostler
EQUIPMENT: 5 tone excavator
ESTIMATED GROUND m (m AHD):

DEPTH (m): 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

ELEVATION (mAHD): 387.2, 387.0, 386.8, 386.6, 386.4, 386.2, 386.0, 385.8, 385.6, 385.4, 385.2, 385.0, 384.8, 384.6, 384.4, 384.2, 384.0, 383.8, 383.6, 383.4, 383.2, 383.0, 382.8

GRAPHIC	DESCRIPTION	DENSITY CONSIST. STRENGTH	LAYER
SM	TOPSOIL: Silty SAND, very dark grey, poorly sorted, fine grained sand, with clay, trace gravel, roots	loose	3
ML	Gravelly SILT, very dark olive brown, poorly sorted, low plasticity, trace sand, roots gravel	firm	5
CI	Silty Gravelly CLAY with sand, olive brown, well sorted, medium plasticity angular gravel	firm to stiff	7
CI	Silty Gravelly CLAY trace sand, olive yellow, well sorted, medium plasticity angular gravel	stiff to hard	9
CH	Sandy CLAY trace gravel, yellow, high plasticity, medium to coarse grained sand		10
CH	CLAY with sand, trace gravel, yellow, high plasticity	firm	12
CH	CLAY with sand, trace gravel, yellowish brown, high plasticity angular gravel	stiff to very stiff	13
CH	CLAY with sand, trace gravel, yellowish brown, high plasticity	stiff	14
CH	INFERRED CLAY with sand, trace gravel, yellowish brown, high plasticity	stiff to very stiff	14

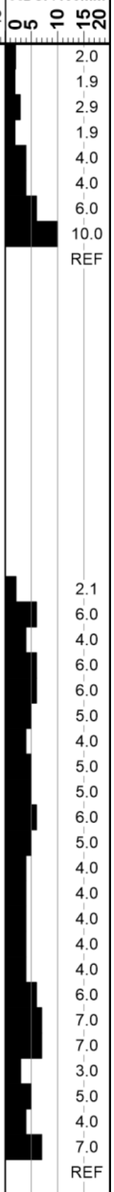
GP Bucket Ended at Target Depth at 3.3 m in stiff yellowish brown CLAY with sand, trace gravel
End of Test Pit at 3.3m depth.

MOISTURE Index: Wet; Moist; Wet

MOISTURE %	SAMPLE
36	DS
18	DS
17	DS
14	DS
22	DS
23	DS
27	DS
39	DS

MOISTURE Well; TEST; Cu (kPa); UCS (kg/cm²)

(IS50 MPa) (CBR) NSPT	NDCP/100mm
(3)	2.0
(3)	1.9
(4)	2.9
(3)	1.9
(6)	4.0
(6)	4.0
(9)	6.0
(20)	10.0
(REF)	REF
(3)	2.1
(9)	6.0
(6)	4.0
(9)	6.0
(9)	6.0
(7)	5.0
(6)	4.0
(7)	5.0
(7)	5.0
(9)	6.0
(7)	5.0
(6)	4.0
(6)	4.0
(6)	4.0
(6)	4.0
(6)	4.0
(9)	6.0
(12)	7.0
(12)	7.0
(4)	3.0
(7)	5.0
(6)	4.0
(12)	7.0
(REF)	REF



GROUNDWATER: Not Encountered

PAGE 1 of 1

TESTING: Penetrometer: AS 1289.6.3.2

DCP Blows per 100mm. For penetrometer blows per 100mm <1, distance travelled per blow is measured and converted back to blows per 100mm

DS: disturbed sample; PV: pocket vane; PP: pocket penotrometer; FV(Ømm): downhole field vane; U50: undisturbed 50mm sample; REF: DCP refusal

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Positioning: GDA94 & mAHD

ASSESSMENT: Geotechnical Site Investigation
STRUCTURE: Extension
EASTING: 519262.2
NORTHING: 5244779.1
ACCURACY
HORIZ: 0.1m **VERT:** 0.1m

Test Pit : TP02

DATE TESTED: 23/03/2026
LOGGED BY: Y. Karki
ELEVATION: 384.3

LOCATION: 40 Wolfes Road - Neika
CLIENT: Hamish and Rachel Ostler

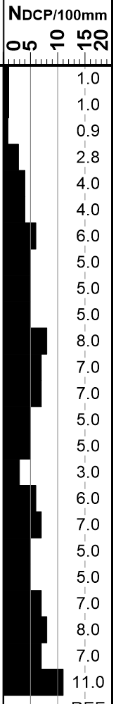
EQUIPMENT: 5 tone excavator
ESTIMATED GROUND m (m AHD):

DEPTH (m)	GRAPHIC	DESCRIPTION	DENSITY CONSIST. STRENGTH	LAYER	MOISTURE Index	%	Well	SAMPLE	TEST	Cu (kPa)	UCS (kg/cm²)
0.0	CI	Silty Sandy CLAY trace gravel, very dark greyish brown, mottled yellowish brown, medium plasticity, medium to coarse grained sand	soft	4	Wet	40		DS			
	CH	Sandy CLAY, olive yellow, poorly sorted, high plasticity, medium to coarse grained sand	soft to stiff	8		26		DS			
0.5	CH	Sandy CLAY trace gravel, yellow, high plasticity, medium to coarse grained sand	stiff to very stiff	10	Moist						
1.0	CH	CLAY with sand, trace gravel, yellowish brown, high plasticity		11	Wet						
2.0	CH	CLAY with sand, trace gravel, yellow, high plasticity	very stiff to hard	12	Moist						

ELEVATION (mAHD): 384.2, 384.0, 383.8, 383.6, 383.4, 383.2, 383.0, 382.8, 382.6, 382.4, 382.2, 382.0, 381.8, 381.6

DEPTH (m): 0.0, 0.5, 1.0, 1.5, 2.0, 2.5

Borehole Ended At Target Depth
End of Test Pit at 2.7m depth.



(IS50 MPa) (CBR) NSPT	NDCP/100mm
(1)	1.0
(1)	1.0
(1)	0.9
(4)	2.8
(6)	4.0
(6)	4.0
(9)	6.0
(7)	5.0
(7)	5.0
(7)	5.0
(14)	8.0
(12)	7.0
(12)	7.0
(7)	5.0
(7)	5.0
(4)	3.0
(9)	6.0
(12)	7.0
(7)	5.0
(7)	5.0
(12)	7.0
(14)	8.0
(12)	7.0
(15)	11.0
(REF)	REF

GROUNDWATER: Not Encountered

PAGE 1 of 1

TESTING: Penetrometer: AS 1289.6.3.2

DCP Blows per 100mm. For penetrometer blows per 100mm <1, distance travelled per blow is measured and converted back to blows per 100mm

DS: disturbed sample; PV: pocket vane; PP: pocket penotrometer; FV(Ømm): downhole field vane; U50: undisturbed 50mm sample; REF: DCP refusal

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Positioning: GDA94 & mAHD

ASSESSMENT: Geotechnical Site Investigation
STRUCTURE: Extension
EASTING: 519268.5
NORTHING: 5244775.1
ACCURACY
HORIZ: 0.1m **VERT:** 0.1m

Point : PT03
DATE TESTED: 23/03/2026
LOGGED BY: Y. Karki
ELEVATION: 384.1

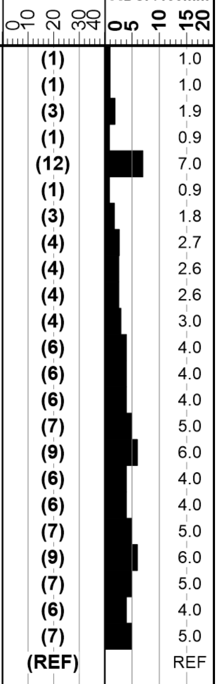
LOCATION: 40 Wolfes Road - Neika
CLIENT: Hamish and Rachel Ostler

EQUIPMENT: 5 tone excavator
ESTIMATED GROUND m (m AHD):

DEPTH (m)	GRAPHIC	DESCRIPTION	DENSITY CONSIST. STRENGTH	LAYER	ELEVATION (mAHD)	MOISTURE Index	MOISTURE %	MOISTURE Well	SAMPLE	TEST	Cu (kPa)	UCS (kg/cm²)
0.0	CH	FILL: CLAY with sand, trace gravel, brownish yellow, high plasticity		1	384.0 383.8	Wet	37		DS			
0.5 1.0 1.5 2.0	CH	INFERRED CLAY	soft to very stiff	2	383.6 383.4 383.2 383.0 382.8 382.6 382.4 382.2 382.0 381.8							

DCP Terminated at 2.4 m Depth
DCP Profiling Only

(IS50 MPa) (CBR) NSPT	NDCP/100mm
(1)	1.0
(1)	1.0
(3)	1.9
(1)	0.9
(12)	7.0
(1)	0.9
(3)	1.8
(4)	2.7
(4)	2.6
(4)	2.6
(4)	3.0
(6)	4.0
(6)	4.0
(6)	4.0
(7)	5.0
(9)	6.0
(6)	4.0
(6)	4.0
(7)	5.0
(9)	6.0
(7)	5.0
(6)	4.0
(7)	5.0
(REF)	REF



GROUNDWATER: NA

PAGE 1 of 1

TESTING: Penetrometer: AS 1289.6.3.2

DCP Blows per 100mm. For penetrometer blows per 100mm <1, distance travelled per blow is measured and converted back to blows per 100mm

DS: disturbed sample; PV: pocket vane; PP: pocket penotrometer; FV(Ømm): downhole field vane; U50: undisturbed 50mm sample; REF: DCP refusal

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Positioning: GDA94 & mAHD

ASSESSMENT: Geotechnical Site Investigation
STRUCTURE: Extension
EASTING: 519276.9
NORTHING: 5244770.5
ACCURACY
HORIZ: 0.1m **VERT:** 0.1m

Test Pit : TP04

DATE TESTED: 23/03/2026
LOGGED BY: Y. Karki
ELEVATION: 381.6

LOCATION: 40 Wolfes Road - Neika
CLIENT: Hamish and Rachel Ostler

EQUIPMENT: 5 tone excavator
ESTIMATED GROUND m (m AHD):

DEPTH (m)	GRAPHIC	DESCRIPTION	DENSITY CONSIST. STRENGTH	LAYER	MOISTURE Index	%	Well	SAMPLE	TEST	Cu (kPa)	UCS (kg/cm²)
0.0	ML	Gravelly SILT, very dark olive brown, poorly sorted, low plasticity, trace sand, roots gravel	firm	5	M ois t						
	CH	Sandy CLAY trace gravel, strong brown, high plasticity, medium to coarse grained sand	stiff	6	Wet	24		DS			
0.5	CH	Sandy CLAY, olive yellow, poorly sorted, high plasticity, medium to coarse grained sand	firm to stiff	8							
1.0	CH	Sandy CLAY trace gravel, yellow, high plasticity, medium to coarse grained sand		10	Moist						
1.5	CH	CLAY with sand, trace gravel, yellow, high plasticity	stiff to very stiff	12							
2.0	CH	CLAY with sand, trace gravel, yellowish brown, high plasticity angular gravel	stiff to hard	13	Wet						
2.5	CH	CLAY with sand, trace gravel, yellowish brown, high plasticity	hard	14							
	CI			15		31		DS			
3.0	CI	SOIL & COBBLES: Silty Gravelly CLAY trace sand, yellow, well sorted, medium plasticity angular gravel; 50% SANDSTONE cobbles	very stiff to hard	15							
		INFERRED SOIL & COBBLES: Silty Gravelly CLAY trace sand, yellow, well sorted, medium plasticity angular gravel; 50% SANDSTONE cobbles									

ELEVATION (mAHD): 381.6, 381.4, 381.2, 381.0, 380.8, 380.6, 380.4, 380.2, 380.0, 379.8, 379.6, 379.4, 379.2, 379.0, 378.8, 378.6, 378.4, 378.2

(IS50 MPa) (CBR) NSPT 0 10 20 30 40	NDCP/100mm 0 5 10 15 20
(3)	2.0
(6)	4.0
(4)	2.9
(4)	2.8
(3)	1.8
(3)	1.8
(3)	1.8
(2)	1.7
(2)	1.7
(4)	2.6
(4)	2.5
(4)	2.5
(4)	2.5
(6)	4.0
(3)	2.4
(6)	4.0
(7)	5.0
(9)	6.0
(7)	5.0
(7)	5.0
(6)	4.0
(6)	4.0
(15)	10.0
(15)	12.0
(15)	12.0
(12)	7.0
(12)	7.0
(15)	20.0
(15)	14.0
(20)	12.0
(14)	8.0
(12)	7.0
(12)	7.0
(20)	20.0
(REF)	REF

GP Bucket Ended at Target Depth at 3 m in hard yellow SOIL & COBBLES: Silty Gravelly CLAY trace sand
End of Test Pit at 3m depth.

GROUNDWATER: Encountered at 0.1 m Below Ground Surface

PAGE 1 of 1

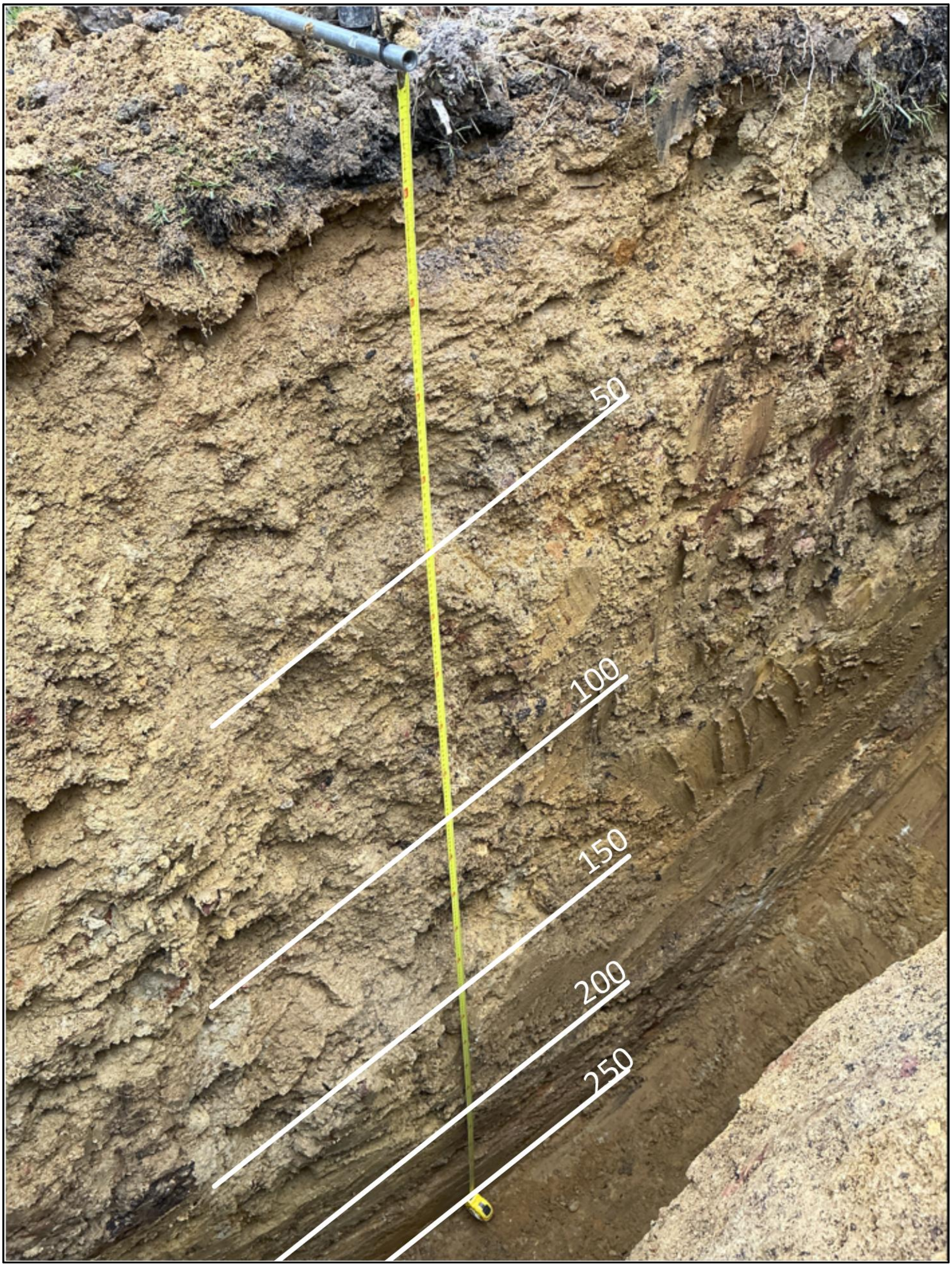
TESTING: Penetrometer: AS 1289.6.3.2

DCP Blows per 100mm. For penetrometer blows per 100mm <1, distance travelled per blow is measured and converted back to blows per 100mm

DS: disturbed sample; PV: pocket vane; PP: pocket penotrometer; FV(Ømm): downhole field vane; U50: undisturbed 50mm sample; REF: DCP refusal

Appendix C Core Photographs

TP01



TP02

TP04

Appendix D Explanatory Notes

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USCS Soil Classification Methodology

Soil classification was undertaken in accordance with the Unified Soil Classification System (USCS) and AS 1726 – Geotechnical Site Investigations, using a combination of particle size distribution and plasticity assessment. This process was applied consistently to all soil layers encountered.

1. Particle Size Distribution (Wet Sieve Analysis)

Particle size analysis was performed by wet sieving in accordance with Australian Standard sieve sizes:

- Gravel fraction: >2.36 mm
- Sand fraction: 0.075 mm to 2.36 mm
- Fines fraction (silt + clay): <0.075 mm

Samples were soaked (often overnight) to fully disperse fines prior to sieving. Wet sieving is particularly effective for Tasmanian soils, which are often dispersive, ensuring accurate quantification of the fines fraction. The oversize fraction (>63 mm) was excluded from the mass percentages before classification.

2. Plasticity Assessment

Plasticity of the fines fraction was determined using:

- Laboratory Atterberg limits, where available, with liquid limit (WL) and plasticity index (PI) plotted on the Plasticity Chart (AS 1726) to determine the fines classification (silt vs clay) and plasticity level (low, medium, high).
- Field index tests (where Atterberg limits were not available), following Table 1 & Table 2:
  - Dry strength – resistance of dried soil to crushing.
  - Dilatancy – reaction of a moist soil pat to shaking.
  - Toughness – resistance of a soil thread near the plastic limit.

Table 1 Field Assessment of Fine-Grained Soils (adapted from AS 1726 Table 7)

Dry Strength		**Dilatancy (reaction to shaking)**		**Toughness (consistency near plastic limit)**	
None	The dry specimen crumbles into powder with mere pressure of handling.	None	No visible reaction or change in the specimen.	Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.
Low	The dry specimen crumbles into powder with some finger pressure.	Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear during squeezing.		
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure.			Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness.
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface.	Rapid	Water appears quickly on the surface of the specimen during shaking and disappears during squeezing.	High	Considerable pressure is required to roll the thread near the plastic limit. The thread and the lump have very high stiffness.
Very High	The dry specimen cannot be broken between the thumb and a hard surface.				

Table 2 Identification of Fine-Grained Soils by Visual–Tactile Methods (adapted from AS 1726 Table 8)

Soil description	Identification of inorganic fine-grained soils		
	Dry Strength	**Dilatancy**	**Toughness and Plasticity**
SILT	None to low	Slow to rapid	Low or thread cannot be formed
Clayey SILT — Clay/silt mixtures of low plasticity	Low to medium	None to slow	Low to medium
Silty CLAY — Silt/clay mixtures of medium plasticity	Medium to high	None to slow	Medium
High plasticity CLAY	High to very high	None	High

3. Classification Hierarchy

3.1 Fine- vs Coarse-Grained determination

- Fine-grained soils: More than 35% (by mass) passes the 0.075 mm sieve → classify using Table 3
- Coarse-grained soils: More than 65% (by mass) is retained on the 0.075 mm sieve → classify using Table 4.

3.2 Coarse-grained soils (Table 4):

1. **Determine Gravel vs Sand:**
    - Gravel (G*) – more than 50% of the coarse fraction is retained on the 2.36 mm sieve.
    - Sand (S*) – less than 50% of the coarse fraction is retained on the 2.36 mm sieve.
2. **Assign fines modifiers:**
    - ≤5% fines: “Clean” gravels/sands (GW, GP, SW, SP).
    - 5–12% fines: Dual classification (e.g., SP-SM, GW-GM).
    - ≥12% fines: Silty or clayey modifiers (GM, GC, SM, SC) based on fines plasticity from Atterberg limits or field index tests.

3.3 Fines classification for coarse-grained soils

When coarse-grained soils contain ≥12% fines, the fines fraction is classified as silty or clayey based on:

- Atterberg limits where available; or
- Field index tests (Table 1 & Table 2) where Atterberg limits are not available.

3.4 Fine-grained soils (Table 3)

Fine-grained soils are those with >35% (by mass) passing the 0.075 mm sieve.

- With Atterberg limits available: WL and PI are plotted on the Plasticity Chart (AS 1726) to determine plasticity level (low, medium, or high) and USCS classification (ML, CL/CI/CH, MH, OL, OH).
- Where Atterberg limits are not available: The fines are classified directly in accordance with Table 3 by comparing field index test results (dry strength, dilatancy, toughness) to the criteria given for each USCS group. This allows direct assignment of ML, CL/CI/CH, MH, or OL/OH without reference to the A-line.

Organic soils (OL, OH) are identified based on colour, odour, and fibrous texture in addition to field index characteristics.

4. Integration of Results

The final USCS group symbol for each layer was determined by integrating:

- The proportion of gravel, sand, and fines from wet sieve analysis.
- The classification of the fines fraction using either Atterberg limits or field index methods.
- The classification hierarchy in Table 3 & Table 4.

This combined approach ensures that soil classification is both quantitatively accurate and fully compliant with AS 1726, while allowing consistent classification whether laboratory Atterberg limit testing is available.

Table 3 Classification of Fine-Grained Soils (adapted from AS 1726 Table 10)

Major Division	Group Symbol	Typical names	Field classification of silt and clay			Laboratory classification
			Dry strength	Dilatancy	Toughness	% <0.075
SILT and CLAY (low to medium plasticity, %)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
	CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
	OL	Organic silt	Low to medium	Slow	Low	Below A line
SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
	CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
	OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
Highly organic soil	Pt	Peat, highly organic soil	—	—	—	—

Table 4 Classification of Coarse-Grained Soils (adapted from AS 1726 Table 9)

Major Division	Group Symbol	Typical names	Field classification of sand and gravel	Laboratory classification
GRAVEL (more than half of coarse fraction is larger than 2.36 mm)	GW	Gravel and gravel–sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤5% fines, Cu > 4, 1 < Cc < 3
	GP	Gravel and gravel–sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤5% fines, fails to comply with above
	GM	Gravel–silt mixtures and gravel–sand–silt mixtures	‘Dirty’ materials with excess of non-plastic fines, zero to medium dry strength	≥12% fines, fines are silty

Major Division	Group Symbol	Typical names	Field classification of sand and gravel	Laboratory classification
	GC	Gravel–clay mixtures and gravel–sand–clay mixtures	'Dirty' materials with excess plastic fines, medium to high dry strength	≥12% fines, fines behave as clay
SAND (more than half of coarse fraction is smaller than 2.36 mm)	SW	Sand and gravel–sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤5% fines, Cu > 6, 1 < Cc < 3
	SP	Sand and gravel–sand mixtures, little or no fines, uniform sands	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤5% fines, fails to comply with above
	SM	Sand–silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥12% fines, fines are silty
	SC	Sand–clay mixtures	'Dirty' materials with excess plastic fines, medium to high dry strength	≥12% fines, fines are clayey

Standard Methodology for Determination of Soil Reactivity and Index of Shrink–Swell (Ips) for SIFE Investigations

1. Introduction

This methodology outlines the procedures adopted by Enviro-Tech Consultants Pty. Ltd. for determining soil reactivity and deriving the Index of Shrink–Swell (Ips) for each soil layer in accordance with the principles of AS 2870. The method combines Australian Standard testing procedures with enhanced correlation techniques developed from an extensive dataset of over 2,000 field and laboratory tests.

The approach ensures consistent, accurate classification of soil reactivity across a wide range of soil types. By combining standard and modified testing procedures, it enables calculation of profile movement for complex soil profiles, accounting for groundwater levels, bedrock depth, and particle size distribution.

2. Sampling and Preparation

2.1 Undisturbed Sampling

Undisturbed samples are collected using a thin-wall sampler to preserve natural soil structure and in-situ moisture conditions when performing shrink–swell testing. A 45 mm diameter core sampler is used for these tests to ensure uniformity and comparability between results. Most other laboratory testing is carried out on disturbed samples, which is one of the advantages of the linear shrinkage and modified linear shrinkage testing methods.

2.2 Sample Identification

All samples are assigned a Unified Soil Classification System (USCS) code using accurate laboratory and field identification techniques in accordance with AS 1726 (detailed procedure included herein). This classification underpins the correlation methods described in later sections.

2.3 Moisture Content Measurement

Field moisture content is recorded at the time of sampling, providing baseline data for correlation to laboratory shrink–swell results.

3. Standard Testing Procedures

3.1 Shrink–Swell Testing

Shrink–swell testing is performed on cohesive soils in accordance with the relevant Australian Standard method for determining the shrink–swell index. This test provides the primary Ips value for these soil types.

3.2 Linear Shrinkage Testing

Linear shrinkage testing is carried out in accordance with the Australian Standard method, which determines shrinkage from a soil prepared at its liquid limit. This standard approach typically excludes a proportion of the sandy fraction.

4. Secondary Modified Linear Shrinkage Method

4.1 Rationale

In practice, the relationship between shrink–swell test results and standard linear shrinkage results is often inconsistent, particularly for non-cohesive or marginally cohesive soils. To improve correlation, a secondary modified linear shrinkage method has been developed.

4.2 Modified Moisture Basis

Instead of preparing samples solely at the liquid limit, this method uses a “modified moisture” content representative of upper-range field moisture conditions for each USCS soil type. These values are derived from a dataset of over 2,000 samples collected predominantly during winter or immediately thereafter, representing the highest seasonal moisture levels without crossing into “abnormal moisture conditions” as defined in AS 2870.

4.3 Application to Non-Cohesive Soils

This approach enables reactivity assessment of sandy and silty soils that are unsuitable for shrink–swell testing due to their inability to remain intact during testing, but which still display measurable reactivity.

5. Gravel and Cobble Fraction Adjustments

5.1 Gravel Fraction

For all materials, the sand fraction is retained in testing, and the gravel fraction is re-added into the calculation. Because gravel has negligible moisture absorption, its proportion is used to adjust shrinkage values downwards.

5.2 Cobbles

Where cobbles are present:

- 0–35% cobbles: shrinkage is scaled according to the proportion of soil matrix between cobbles.
- 35% cobbles: shrinkage is considered negligible, as the soil matrix is insufficient to impart meaningful reactivity.

6. Correlation and Calibration

6.1 Dataset Development

Extensive correlation has been undertaken between:

- Standard shrink–swell results
- Standard linear shrinkage results
- Modified linear shrinkage results

6.2 USCS-Based Correlation

Accurate USCS classification is the key input variable. Once correlations are established for each USCS class, Ips values can be assigned to future samples based solely on classification and moisture parameters, without requiring repeated shrink–swell testing.

7. Predictive Modelling and Database Search Method

Enviro-Tech Consultants maintains a large and continuously expanding database of soil test results, including shrink–swell, linear shrinkage, particle size distribution, USCS classification, and detailed field descriptions (e.g., colour, texture, structure).

When assessing a new Site, we search this database for comparable sites using multiple parameters:

- Geology – parent material type and origin
- USCS classification – precise laboratory classification

- Soil colour and descriptive features – matching field logging records
- Particle size distribution – percentage gravel, sand, and fines

This multi-parameter search allows us to identify highly similar soils and adopt Ips values from past testing at those sites with confidence. The approach reduces the need for repeated shrinkage or shrink–swell testing where soils are well represented in the database, while still meeting the requirements for reliable reactivity estimation.

8. Compliance with AS 2870 – Clause 2.3.2 (C2 & C3)

Our predictive approach aligns directly with the requirements of AS 2870 Clause 2.3.2:

c (ii): We maintain and utilise a database of past test results to estimate soil reactivity for sites with similar soil and geological conditions.

C (iii): Testing is repeated at regular intervals to ensure correlations remain valid. At minimum, reactivity testing is conducted once every 50 sites, but in practice we test far more frequently – typically at least once every 20 sites, and rarely more than six months between tests. On average, new verification testing is undertaken approximately monthly.

This compliance ensures that our methods are both technically robust and standards-compliant, providing clients with defensible, high-quality results.

9. Calculation of Profile Movement

9.1 Ips Values per Layer

An Ips value is determined for each soil layer based on test results or correlations.

9.2 Adjustment for Groundwater and Bedrock

Where groundwater or bedrock occurs within the profile, Ips values are reduced for the affected layers in accordance with AS 2870 principles.

9.3 Design Suction Change Depth (Hs)

Given the lack of statewide, high-resolution climatic data for Tasmania, a conservative Hs value of 3.0 m is adopted for all sites, in preference to regionalised values. This ensures a cautious approach where actual depth of suction change cannot be accurately modelled.

9.4 Surface Suction Change (Δus)

A standard surface suction change of 1.2 is applied in calculations, in line with AS 2870.

10. Advantages of the Modified Method

- Allows for reactivity assessment across all soil types, including non-cohesive sands and silts.
- Provides consistent correlation between laboratory and field methods.
- Enables accurate whole-profile movement estimation based on standardised USCS classification.
- Incorporates gravel and cobble fraction corrections for more realistic movement predictions.
- Reduces reliance on repeat laboratory shrink–swell testing for every sample.
- Fully compliant with AS 2870 Clause 2.3.2 (C2 & C3).

11. Limitations

- The method assumes accurate USCS classification and field moisture determination.
- The modified linear shrinkage method requires prior calibration for each USCS type.
- Adoption of a conservative Hs value may slightly overestimate movement in some locations.

12. Conclusion

This methodology blends rigorous Australian Standard test procedures with enhanced, data-driven correlation techniques, enabling Enviro-Tech Consultants Pty. Ltd. to deliver accurate, consistent, and site-specific soil reactivity assessments across Tasmania. The inclusion of >2,000 test results, gravel and cobble adjustments, predictive modelling from a comprehensive database, and modified moisture testing provides a robust basis for predicting profile movement in varied geological conditions, while maintaining strict compliance with AS 2870.

Appendix E Soil and Rock Testing

Dynamic Cone Penetrometer (DCP)

Dynamic cone penetrometer (DCP) testing was conducted according to AS 1289.6.3.2 with the results presented in Appendix B.

Soil Characterisation

Table 3 summarises the soil classification results for each layer encountered, including particle size distribution, plasticity assessment, and the assigned USCS group symbol.

Classifications were undertaken in accordance with AS 1726 – Geotechnical Site Investigations using the methodology provided in the Explanatory Notes section of this report.

Particle size distributions were determined by wet sieve analysis, and fines classifications were based on Atterberg limits where available, or on field index tests (dry strength, dilatancy, toughness) in accordance with AS 1726 Tables 7, 8, 9, and 10.

Full explanatory notes and reference tables are provided in Explanatory Notes section of this report.

Table 3 Summary of the Soil Characterisation

Layer	Soil	Borehole	Depth From (m)	Field Moisture %	Gravel %	Sand %	Fine %	Analysed Plasticity	Assigned USCS
1	CLAY	PT03	0	37.4	3.8	29.8	66.4	H	CH
3	Silty SAND	TP01	0	35.8	3	70.9	26.1		SM
4	Silty Sandy CLAY	TP02	0.1	40.2	11	35.2	53.8	M	CI
5	Gravelly SILT	TP01	0.1	17.6	46.6	13.2	40.2	L	ML
6	Sandy CLAY	TP04	0.1	23.9	4.2	35.7	60.1	H	CH
7	Silty Gravelly CLAY	TP01	0.2	16.6	45.1	18.4	36.5	M	CI
8	Sandy CLAY	TP02	0.3	26.2	2.3	47.2	50.5	H	CH
9	Silty Gravelly CLAY	TP01	0.4	14.4	52.6	9.9	37.5	M	CI
10	Sandy CLAY	TP01	1.5	22.1	9	36.5	54.5	H	CH
12	CLAY	TP01	1.9	22.8	8.7	30.7	60.6	H	CH
13	CLAY	TP01	2.2	27.1	14.5	28.3	57.2	H	CH
14	CLAY	TP01	3.2	38.6	3	19	78	H	CH
15	Silty Gravelly CLAY	TP04	2.9	30.9	56.4	9.9	33.7		GC

Soil Profiles

The geology of the site has been documented and described according to Australian Standard AS1726 for Geotechnical Site Investigations, which includes the Unified Soil Classification System (USCS). Soil layers, and where applicable, bedrock layers, are summarized in Table 4.

Table 4 Soil Summary Table

#	Layer	Details	USCS	TP01	TP02	PT03	TP04
1	Silty SAND	TOPSOIL: Silty SAND, very dark grey, poorly sorted, fine grained sand, with clay, trace gravel, roots, L	SM	0-0.1 DS@0.0			
2	Silty Sandy CLAY	Silty Sandy CLAY trace gravel, very dark greyish brown, mottled yellowish brown, medium plasticity, medium to coarse grained sand, S	CI		0-0.2 DS@0.1		
3	Gravelly SILT	Gravelly SILT, very dark olive brown, poorly sorted, low plasticity, trace sand, roots; gravel, F	ML	0.1-0.2 DS@0.1			0-0.1
4	Sandy CLAY	Sandy CLAY trace gravel, strong brown, high plasticity, medium to coarse grained sand, St	CH				0.1-0.3 DS@0.1
5	Silty Gravelly CLAY	Silty Gravelly CLAY with sand, olive brown, well sorted, medium plasticity; angular gravel, F-St	CI	0.2-0.4 DS@0.2			
6	Sandy CLAY	Sandy CLAY, olive yellow, poorly sorted, high plasticity, medium to coarse grained sand, S-St	CH		0.2-0.5 DS@0.3		0.3-0.8
7	Silty Gravelly CLAY	Silty Gravelly CLAY trace sand, olive yellow, well sorted, medium plasticity; angular gravel, St-H	CI	0.4-1.5 DS@0.4			
8	Sandy CLAY	Sandy CLAY trace gravel, yellow, high plasticity, medium to coarse grained sand, F-VSt	CH	1.5-1.9 DS@1.5	0.5-0.9		0.8-1.6
9	CLAY	CLAY with sand, trace gravel, yellowish brown, high plasticity, St-VSt	CH		0.9-2		
10	CLAY	CLAY with sand, trace gravel, yellow, high plasticity, F-H	CH	1.9-2.2 DS@1.9	2-2.7		1.6-2.1
11	CLAY	CLAY with sand, trace gravel, yellowish brown, high plasticity; angular gravel, St-H	CH	2.2-3.2 DS@2.2			2.1-2.8
12	CLAY	CLAY with sand, trace gravel, yellowish brown, high plasticity, St-H	CH	3.2-3.3 DS@3.2 3.3-4.4 INF			2.8-2.9
13	CLAY	CLAY with sand, trace gravel, S-VSt	CH			0-2.3 INF	
14	Silty Gravelly CLAY	SOIL & COBBLES: Silty Gravelly CLAY trace sand, yellow, well sorted, medium plasticity; angular gravel; 50% SANDSTONE cobbles, VSt-H	CI				2.9-3 DS@2.9 3-3.4 INF

Consistency[1]	**VS** Very soft; **S** Soft; **F** Firm; **St** Stiff; **Vst** Very Stiff; **H** Hard. Consistency values are based on soil strengths AT THE TIME OF TESTING and is subject to variability based on field moisture condition
Density[2]	**VL** Very loose; **L** Loose; **MD** Medium dense; **D** Dense; **VD** Very Dense
Rock Strength	**EL** Extremely Low; **VL** Very Low; **L** Low; M Medium; **H** High; **VH** Very High; **EH** Extremely High
DS	Disturbed sample
REF	Borehole refusal
INF	DCP has continued through this layer and the geology has been inferred.

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[1] Soil consistencies are derived from a combination of field index, DCP and shear vane readings.

[2] Soil density descriptions presented in engineering logs are derived from the DCP testing.

Soil Dispersion (Emerson aggregate test)

Select soil samples were tested for dispersion susceptibility using the Emerson Class number method according to AS1289.3.8.1. The results presented in Table 5 demonstrate that:

- Soil tested from the Site is either not dispersive (Class 4 or greater) or only slightly dispersive (Class 3).

Table 5 Summary of the Emerson class results.

Layer	Soil	Depth	Sample ID	Emersion Class	Date Tested	Water	pH
1	FILL: CLAY with sand, trace gravel	0	PT03 0.0	Class 3	8/04/2026	DI 18°C	
3	TOPSOIL: Silty SAND	0	TP01 0.0	Class 8	8/04/2026	DI 18°C	
4	Silty Sandy CLAY trace gravel	0.1	TP02 0.1	Class >4	8/04/2026	DI 18°C	5.3
5	Gravelly SILT	0.1	TP01 0.1	Class 8	8/04/2026	DI 18°C	5.3
6	Sandy CLAY trace gravel	0.1	TP04 0.1	Class 3	8/04/2026	DI 18°C	5.7
7	Silty Gravelly CLAY with sand	0.2	TP01 0.2	Class >4	8/04/2026	DI 18°C	5.5
8	Sandy CLAY	0.3	TP02 0.3	Class 3	8/04/2026	DI 18°C	5.6
9	Silty Gravelly CLAY trace sand	0.4	TP01 0.4	Class >4	8/04/2026	DI 18°C	5.7
10	Sandy CLAY trace gravel	1.5	TP01 1.5	Class >4	8/04/2026	DI 18°C	5.7
12	CLAY with sand, trace gravel	1.9	TP01 1.9	Class >4	8/04/2026	DI 18°C	
13	CLAY with sand, trace gravel	2.2	TP01 2.2	Class >4	8/04/2026	DI 18°C	5.6
14	CLAY with sand, trace gravel	3.2	TP01 3.2	Class >4	8/04/2026	DI 18°C	5.5
15	SOIL & COBBLES: Silty Gravelly CLAY trace sand	2.9	TP04 2.9	Class >4	8/04/2026	DI 18°C	

Soil Aggressivity Testing (Footing Exposure Classification)

Soil samples from across the Site were assessed for potential aggressivity to concrete in accordance with the requirements of AS 2870:2011 – Residential Slabs and Footings (Clauses 5.5.1–5.5.3). Testing was undertaken to determine the salinity exposure class and provide an indicative assessment of sulphate soil potential.

The results are summarised in Table 6 which presents the sampling depth and location, soil texture classification, electrical conductivity (EC1:5), salinity factor (K), calculated saturated extract electrical conductivity (ECe), and the corresponding salinity exposure class (Table 5.1, AS 2870). Soil pH values were also measured and used as a conservative indicator of potential sulphate aggressivity, together with the assigned soil condition class, to derive an indicative sulphate exposure class (Table 5.2, AS 2870).

It is noted that the sulphate assessment has been undertaken on the basis of pH values only, and therefore represents a conservative assumption. Where soils exhibit pH < 5.5 or are otherwise classified within B or C exposure classes, confirmatory laboratory testing of sulphate concentrations may be warranted to refine the exposure classification and confirm appropriate concrete durability requirements.

Salinity testing has been undertaken in accordance with the relevant guidelines and provides a direct basis for assigning salinity exposure classification.

Where aggressive soils are discerned, detailed recommendations for the management of aggressive soils, including concrete strength, curing and reinforcement cover requirements, are presented in Appendix F.

Table 6 Soil Aggressivity Assessment in Accordance with AS 2870:2011

Layer	**Location**	**Depth**	**Saline Soil Determination**					**Sulphate Soil Potential^**		
		From (m)	**USDA Soil Texture Class**	**EC1:5 mS/cm**	**K***	**Ece dS/m**	**Exposure Class**	**pH1:5**	**Soil Condition Class**	**Exposure Class**
1	PT03	0.0	Clay	0.06	5.5	0.33	A1	5.7	B	A1
3	TP01	0.0	Loamy sand	0.08	13.0	1.04	A1	5.3	B	A2^
4	TP02	0.1	Clay loam	0.10	6.5	0.65	A1	5.3	B	A2^
5	TP01	0.1	Silt loam	0.04	8.5	0.34	A1	5.3	B	A2^
6	TP04	0.1	Clay	0.05	5.5	0.28	A1	5.7	B	A1
7	TP01	0.2	Clay loam	0.03	6.5	0.20	A1	5.5	B	A2^
8	TP02	0.3	Sandy clay	0.07	6.0	0.42	A1	5.6	B	A1
9	TP01	0.4	Clay	0.04	5.5	0.22	A1	5.7	B	A1
10	TP01	1.5	Clay	0.05	5.5	0.28	A1	5.7	B	A1
12	TP01	1.9	Clay	0.06	5.5	0.33	A1	5.6	B	A1
13	TP01	2.2	Clay	0.06	5.5	0.33	A1	5.6	B	A1
14	TP01	3.2	Clay	0.08	5.5	0.44	A1	5.5	B	A2^
15	TP04	2.9	Clay	0.07	5.5	0.39	A1	6.0	B	A1

^ Preliminary findings based on soil pH only. Further sulphate testing required to rule out sulphate soil exposure risks

*Electrical conductivity of the 1:5 soil–water extract (EC1:5) was measured at 25 °C and converted to an equivalent saturated paste extract (ECe) using texture-based conversion factors (ECe = k × EC1:5) following Slavich, P.G. & Patterson, R.A. (1990). Estimating the electrical conductivity of saturated paste extracts from 1:5 soil:water suspensions and texture. Australian Journal of Soil Research, 28, 453–463.

Appendix F Geotechnical Interpretation

Interpreted Engineering Ground Model

The detailed soil profile has been simplified into an interpreted engineering ground model for the purpose of preliminary geotechnical assessment and slope stability modelling. This approach has been adopted because the logged soil units are variable and gradational, and the boundaries between materials are not considered to represent laterally continuous discrete stratigraphic layers.

The interpreted model is based primarily on test pit logging, DCP resistance, field consistency, moisture condition, and the relative positions of TP01 and TP04 within the proposed building area. For the purpose of the preliminary slope stability model, the profile has been divided into three engineering units:

- Firm High Plasticity CLAY
- Stiff High Plasticity CLAY
- Very Stiff Medium Plasticity CLAY

The consistency categories used in the model have been interpreted from DCP blow counts recorded per 100 mm penetration. As a general guide, DCP blow counts of approximately 2 blows per 100 mm or less have been interpreted as firm clay, values up to approximately 5 blows per 100 mm have been interpreted as stiff clay, and values up to approximately 9 blows per 100 mm have been interpreted as very stiff clay.

The inferred engineering unit boundaries shown on the cross-section mapped in Figure 2 and illustrated in Figure 3 are interpretive and should not be regarded as sharp geological contacts. They represent interpreted transitions in material behaviour, based on the available investigation data, for the purpose of preliminary assessment and further investigation planning.

The parameters presented in Table 7 are inferred values adopted for preliminary slope stability modelling only. They are based on field observations, DCP-derived strength trends and engineering judgement. No laboratory shear strength testing has been undertaken at this stage. The parameters should therefore be regarded as preliminary and subject to revision following targeted undisturbed sampling and laboratory testing.

Table 7 Adopted Inferred Parameters for Preliminary Slope Stability Modelling

Engineering Unit	Unit Weight γ (kN/m³)	Saturated Unit Weight γsat (kN/m³)	Effective Cohesion c′ (kPa)	Effective Friction Angle ϕ′ (°)	Basis
Firm High Plasticity CLAY	20.5	20.5	5	15	Inferred from DCP resistance, field consistency and moisture condition
Stiff High Plasticity CLAY	20.5	20.5	10	15	Inferred from DCP resistance, field consistency and moisture condition
Very Stiff Medium Plasticity CLAY	17.0	19.0	24	19	Inferred from DCP resistance, field consistency and moisture condition

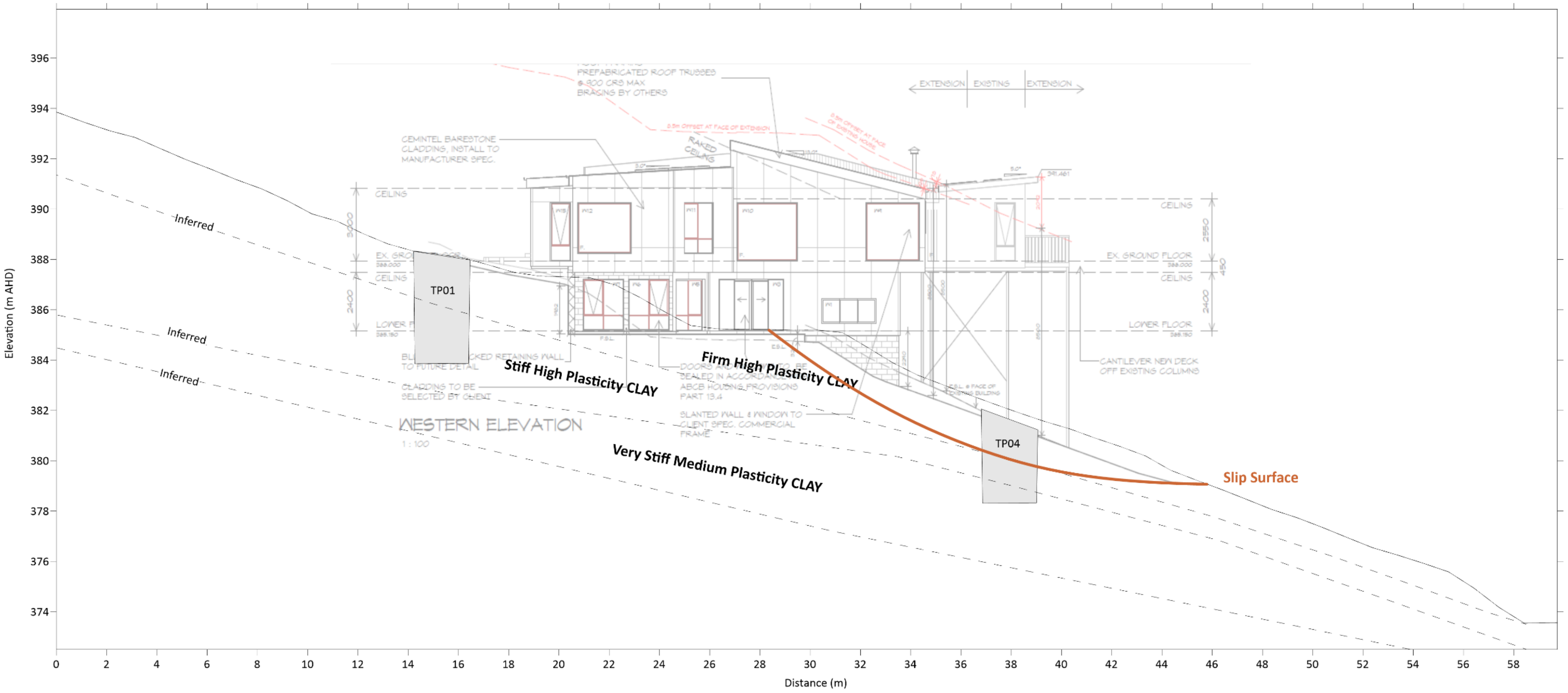


Figure 3 Interpreted Engineering Ground Model and Preliminary Slip Surface. Note: Engineering unit boundaries and slip surface are inferred based on test pit logging, DCP resistance and field observations.

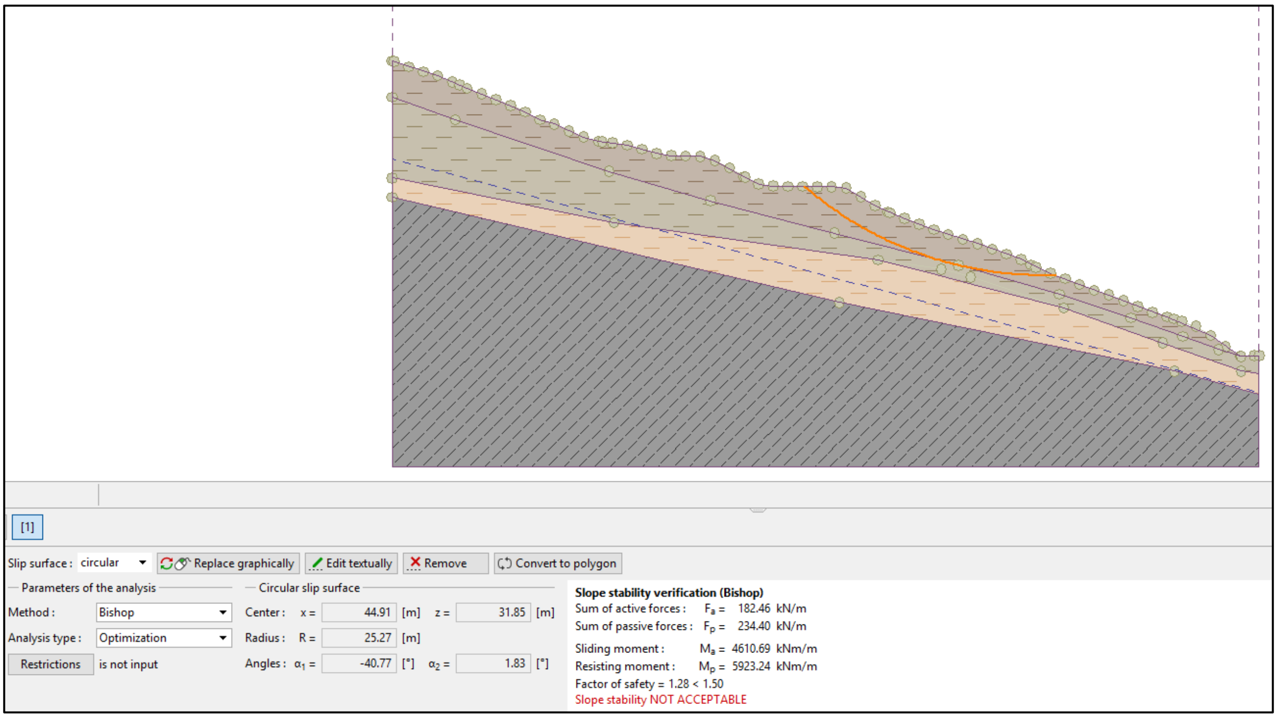
Preliminary Slope Stability Model

A preliminary slope stability model (Figure 4) was undertaken using the interpreted engineering ground model (Figure 3) and the inferred parameters presented in Table 7. The model was developed to assess an indicative shallow slide scenario associated with the proposed earthworks and to determine whether further Site-specific investigation is required.

The model was analysed using the Bishop method. The calculated factor of safety was 1.28, which is below the adopted target factor of safety of 1.50 for long-term stability.

Based on the adopted inferred parameters, the preliminary model indicates that slope stability is marginal and does not meet the adopted screening criterion. However, the model is based on inferred parameters derived from DCP correlations, field observations and engineering judgement only. No laboratory shear strength testing has been undertaken at this stage.

The result should therefore be regarded as a screening-level outcome and not a final land stability assessment. Further targeted sampling and laboratory shear strength testing are required to confirm representative strength parameters and refine the slope stability model.



Slip surface : circular

Parameters of the analysis
Method : Bishop
Analysis type : Optimization
Restrictions is not input

Circular slip surface
Center : x = 44.91 [m] z = 31.85 [m]
Radius : R = 25.27 [m]
Angles : α_1 = -40.77 [°] α_2 = 1.83 [°]

Slope stability verification (Bishop)
Sum of active forces : F_a = 182.46 kN/m
Sum of passive forces : F_p = 234.40 kN/m
Sliding moment : M_a = 4610.69 kNm/m
Resisting moment : M_p = 5923.24 kNm/m
Factor of safety = 1.28 < 1.50
Slope stability NOT ACCEPTABLE

Figure 4 Preliminary slope stability model showing interpreted engineering units and modelled shallow failure mechanism (Bishop Method). The preliminary model returned a calculated factor of safety of 1.28, below the adopted screening criterion of 1.50. Engineering unit boundaries, groundwater conditions and adopted parameters are inferred from DCP resistance trends, field observations and preliminary geotechnical interpretation only.

Footing Minimum Target Depths

Footing design for the proposed structures are to consider the depths of limiting layers at the base of potentially problematic soils. Where practical/allowable, thickened beams may be deepened through problematic soil layers according to engineering specifications (Table 8). Table 9 should be referred to where only 50kPa allowable bearing capacity is required.

Table 8 also presents a summary of the estimated soil depths and associated layers where less than 5mm of vertical soil movement can expected due to soil moisture fluctuations from normal seasonal wetting and drying cycles. Where 5mm tolerances are required, concentrated loads including but not limited to slab edge or internal beam or strip footings shall be supported directly on piers in accordance with minimum target layer depths presented in Table 8, with considerations given to required bearing capacities in accordance with Table 9.

All footing depth, soil movement, and bearing capacity calculations presented in this section are based on interpretive I_{PS} or I_{SS} values derived from field and laboratory data, as outlined in the Explanatory Notes section of this report. These values are used to infer soil reactivity in the absence of direct measurement, in accordance with industry best practice.

Table 8 Soil characteristic surface movements and recommended footing minimum target depths

Footing design parameters	**TP01**	**TP02**	**PT03**	**TP04**
Ys Calculation Depth	0m^	0m^	0m^	0m^
Surface movement Ys (mm)	45	75	75	75
Calculated Equivalent Reactivity Classification (AS2870)	H1	H2	H2	H2
Base of problem soil layer (m)*	0.1	0.3	0.7	0.9
Layer at base of problem soil*	4	8	10	11
Pier/Footing minimum target depth (m)$^{\#}$	>4.5	>3.5	>3.5	>3.5
Allowable bearing capacity at min target depth (kPa)$^{\#}$	150	250	250	250

- No problem layers encountered
^ Calculations relative to surface of borehole at the time of investigation
~ Calculated based on revised soil profile depth/thickness following indicative cut and fill. Inferred fill reactivity indicated (Iss value) which is typically based on more reactive soils expected to be encountered within inferred cut.
* Base of problematic soil layer depth below top of borehole surface at the time of testing to achieve 100 kPa allowable bearing capacity or greater.
# Target soil layer depth where Ys values from normal wetting and drying cycles are estimated at less than 5mm vertical movement. >minimum bored pier depths (see bearing capacity table for bored pier design depths).
The classifications presented in Table 8 represent calculated equivalent reactivity classifications only and should not be interpreted as the governing design classification for the Site. Due to the problematic ground conditions, tree-removal effects, earthworks and slope stability considerations identified during assessment, the Site should be treated as requiring specific engineering design (Class P conditions).

Soil and Rock Allowable Bearing Capacity & End Bearing Capacity

Soil allowable bearing capacity was calculated from correlations with DCP blow counts. A recommended safety factor of 3 is applied in accordance with AS2870. Where high clay and silt content is observed in the soil, soil allowable bearing capacity is determined from undrained shear strengths using field vane correlated DCP values. Interpretive bearing capacity values are presented in Table 9.

Although the DCP testing was conducted through the FILL, there is no interpretation of bearing capacities of the FILL, as it is recommended that the FILL is not used for the support footings due to risk of differential settlement. Unless there is documented evidence that the fill is engineered or engineering confidence that the fill was suitably compaction controlled, building structures should be founded on underlying natural soils or bedrock with suitable bearing capacity for the design loads.

Table 9 Soil allowable bearing capacities and problematic ground conditions.

Depth below investigation surface (m)	**Allowable Bearing Capacity (kPa)**			
	TP01	**TP02**	**PT03**	**TP04**
0	90~	40~	40~	70~
0.1	**100***	50~	50~	170
0.2	**130**	40~	100	130
0.3	**100**	**130***	40~	130
0.4	**170**	**170**	280	90~
0.5	**170**	**170**	40~	90~
0.6	**240**	**240**	90~	90~
0.7	**>400**	**200**	**130***	90~
0.8	**REF**	**200**	**120**	90~
0.9		**200**	**120**	**120***
1		**330**	**140**	**120**
1.1		**280**	**170**	**120**

Depth below investigation surface (m)	Allowable Bearing Capacity (kPa)			
	TP01	TP02	PT03	TP04
1.2		280	170	120
1.3		200	170	170
1.4		200	200	120
1.5		140	240	170
1.6		240	170	200
1.7		280	170	240
1.8		200	200	200
1.9		200	240	200
2		280	200	170
2.1	100	330	170	170
2.2	240	280	200	>400
2.3	170	>400	REF	>400
2.4	240	REF		>400
2.5	240			280
2.6	200			280
2.7	170			>400
2.8	200			>400
2.9	200			>400
3	240			330
3.1	200			280
3.2	170			280
3.3	170			>400
3.4	170			REF
3.5	170			
3.6	170			
3.7	240			
3.8	280			
3.9	280			
4	140			
4.1	200			
4.2	170			
4.3	280			
4.4	REF			

Correlations drawn from DCP and vane shear testing.

REF - Penetrometer Refusal

^ Footings to be founded through the FILL

~ Problematic soil layer attributed to loose, soft, or low allowable bearing capacity soil (<100 kPa)

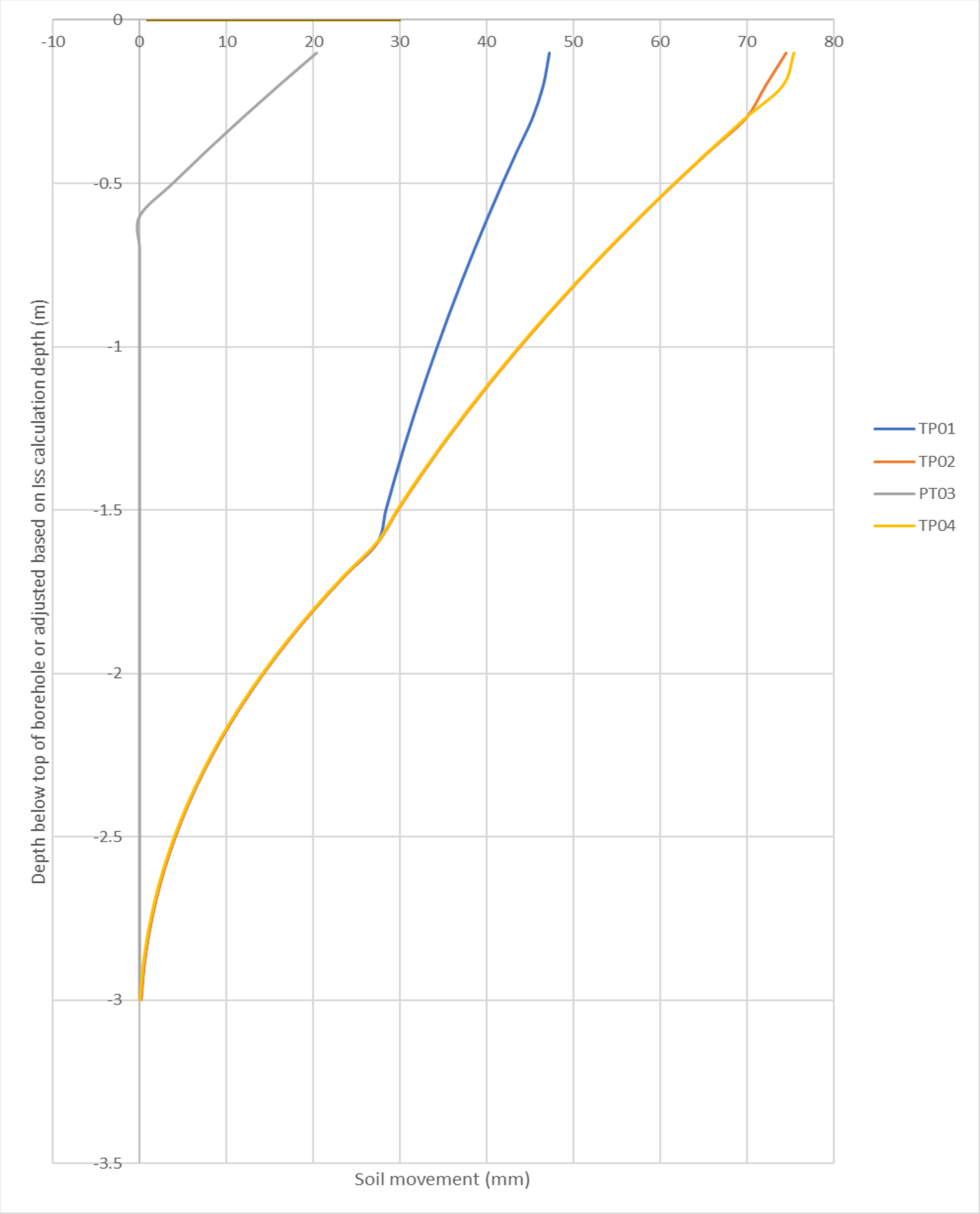
*Soil layer expected at the base of problematic soil layers at test location (or at surface where problematic soils not encountered) to achieve 100 kPa allowable bearing capacity or greater.

Characteristic Soil Movement (Ys)

The characteristic soil movement (soil reactivity) from wetting and drying cycles is calculated according to AS 2870 Section 2.3. The calculations are based on Iss % testing results or correlations with linear shrink data and are based on complete soil profiles for boreholes drilled within the building Site. In the case of where cut and fill are proposed and building finished floor levels (FFL) are made available, the Iss value is recalculated based on the FFL and estimated cut and fill as per Table 8.

According to AS 2870 Section 2.3, calculations consider the depth of groundwater and bedrock. Soil characteristic movements based on lab testing are presented in Figure 5.

Figure 5 Calculated Characteristic Soil Movement Based on Soil Testing



Design Of Footings for Trees

The calculated characteristic surface movement for the Site, excluding tree effects, is Ys = 74.5 mm, rounded to 75 mm, corresponding to Class H2 reactivity.

A large Eucalyptus regnans, approximately 40 m high, was formerly located approximately 3 m from the proposed building envelope. In accordance with Appendix H of AS 2870, the influence distance for a single tree may be taken as 1.0HT, giving an estimated influence distance of approximately 40 m. The ratio Dt/HT is approximately 0.075, which is less than 0.5; therefore, the full tree-induced movement has been adopted for assessment purposes.

For the adopted design suction depth of Hs = 3.0 m, Table H1 gives a maximum design tree drying depth of approximately 3.4 m and a maximum additional suction change of 0.38 pF for a single tree. Application of the Appendix H tree suction profile to the Site soil reactivity profile gives an estimated additional tree-related movement of approximately 25 mm.

The resulting tree-affected characteristic surface movement is therefore estimated to be approximately 100 mm, which exceeds the normal H2 range and is considered consistent with tree-affected Class E footing conditions.

As the tree has reportedly been removed, the relevant design case is considered to be moisture recovery and potential clay heave following tree removal, rather than ongoing tree drying. In accordance with Appendix H of AS 2870, the calculated footing design mound height for the tree-removal case is approximately 75–80 mm.

The footing system should therefore be specifically assessed by the structural engineer for tree-affected reactive clay conditions and the land stability considerations identified within this report.

Concrete Durability Requirements

The soil aggressivity testing results presented in Table 6 have been interpreted in Table 10 to provide indicative requirements for minimum concrete strength, curing duration and reinforcement cover in accordance with AS 2870:2011. This table builds on the previous classification summary by applying the relevant durability provisions to each individual soil layer encountered across the Site.

From these results presented in Table 10, it is generally discerned that in all investigated areas of the Site:

- With an exposure classification of A2, at a minimum, 25 MPa concrete is generally recommended with 40 mm cover using a damp-proofing membrane or 50mm cover without. A minimum curing time of 3 days is recommended.
- It is also essential to refer to other applicable construction standards to ensure compliance with all relevant guidelines and best practices.

Table 10 Interpretation of Soil Aggressivity Results – Minimum Concrete Strength, Curing and Cover

Layer	**Location**	**Depth**	**Exposure Classification**		**Minimum Concrete**	**Minimum Days Curing**	**Cover~**
		From (m)	**Salinity**	**Sulphate^**	**Strength f'c (MPa)^**		
1	TP01	0.0	A1	A2^	20-25^	3	40-50#
2	TP02	0.1	A1	A2^	20-25^	3	40-50#
3	TP01	0.1	A1	A2^	20-25^	3	40-50#
4	TP04	0.1	A1	A1	20	3	40
5	TP01	0.2	A1	A2^	20-25^	3	40-50#
6	TP02	0.3	A1	A1	20	3	40
7	TP01	0.4	A1	A1	20	3	40

Layer	Location	Depth	Exposure Classification		Minimum Concrete	Minimum Days Curing	Cover~
		From (m)	Salinity	Sulphate^	Strength f'c (MPa)^		
8	TP01	1.5	A1	A1	20	3	40
10	TP01	1.9	A1	A1	20	3	40
11	TP01	2.2	A1	A1	20	3	40
12	TP01	3.2	A1	A2^	20-25^	3	40-50#
13	PT03	0.0	A1	A1	20	3	40
14	TP04	2.9	A1	A1	20	3	40

^Sulphate class is conservatively estimated from soil pH and further testing is required on soil samples to confirm if the low pH is attributed to sulphate or other anions within the soil. If pH conditions are attributed primarily to sulphate, then the indicated exposure classification is expected to reliable but subject to sulphate concentration threshold presented in AS2870.

Where a damp-proofing membrane is installed, the minimum reinforcement cover in saline (non A1) soils may be reduced to 30 mm

' Where a damp-proofing membrane is installed, the minimum reinforcement cover in sulphate (non A1) soils may be reduced by 10 mm.

Appendix G Foundation Maintenance & Footing Performance (CSIRO)

Foundation Maintenance and Footing Performance:

A Homeowner's Guide

CSIRO

BTF 18
replaces
Information
Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

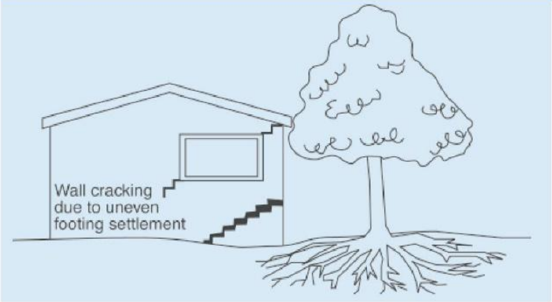
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem.
It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

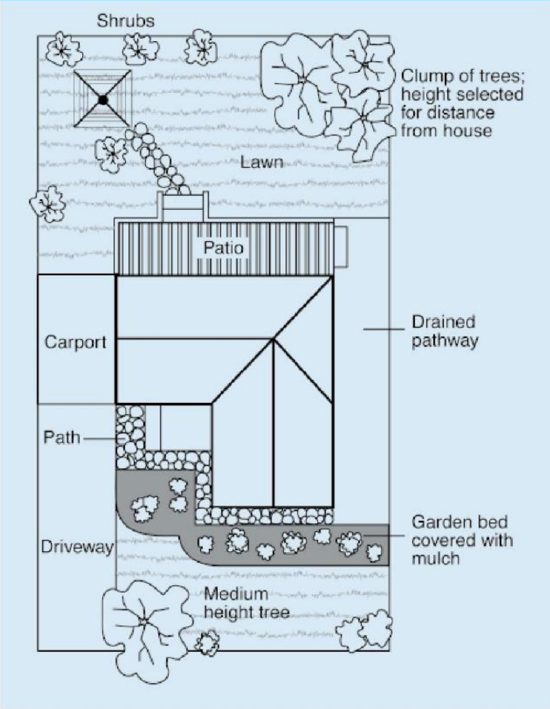
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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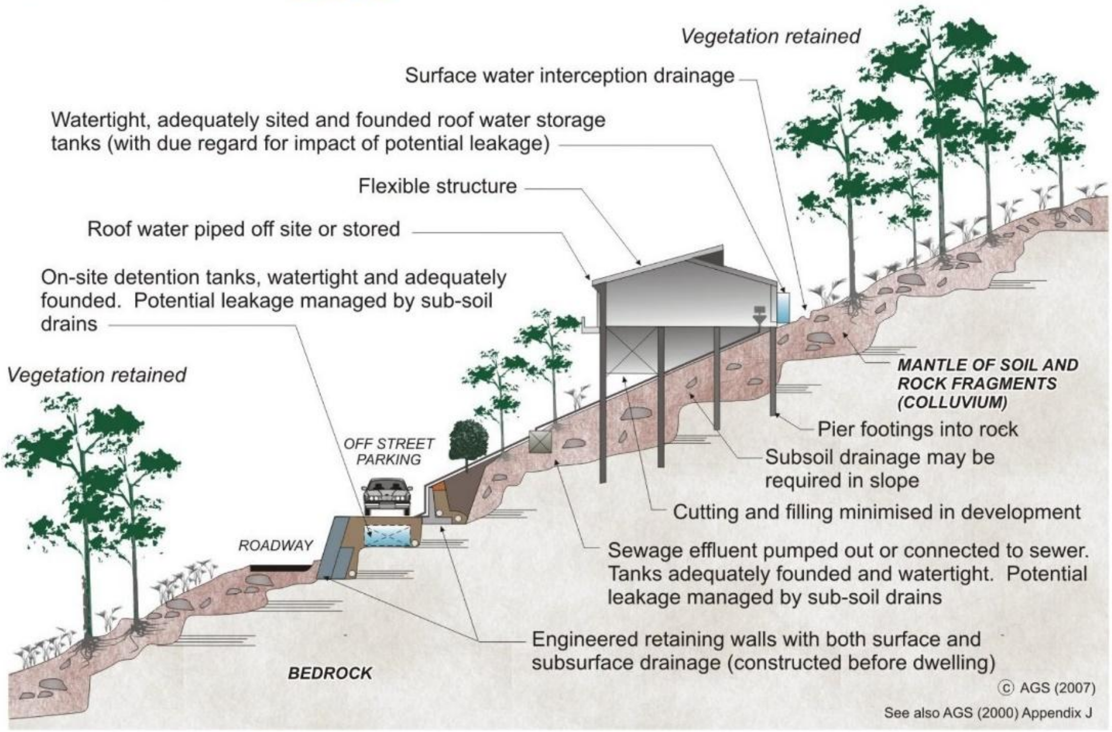
Appendix H Examples of Good Hillside Construction (AGS LRM LR8)

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES OF **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

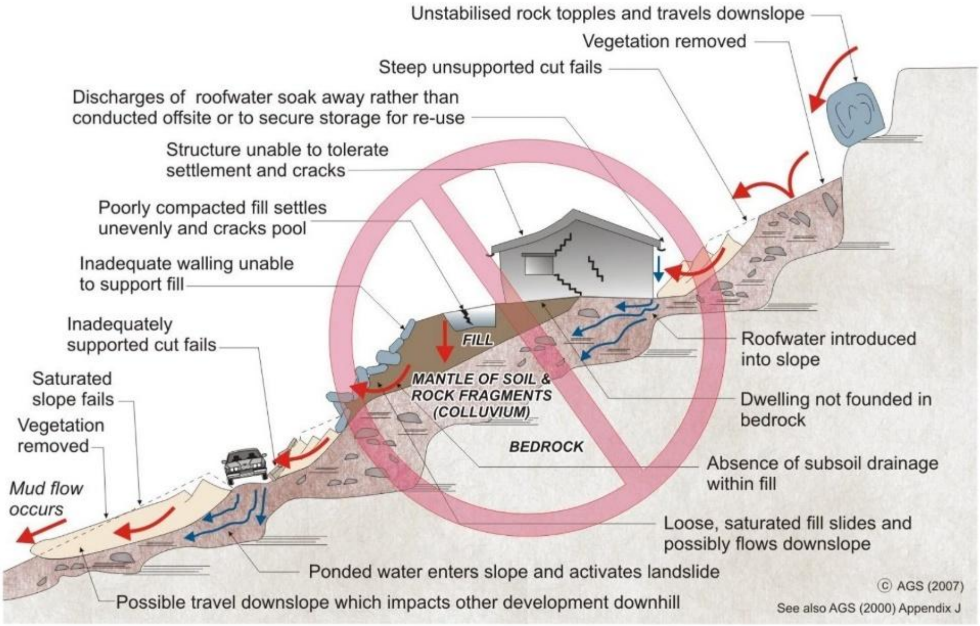
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal

GeoGuide LR10 - Coastal Landslides

- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

CERTIFICATE OF QUALIFIED PERSON – ASSESSABLE ITEM

Section 321

Form **55**

To: Hamish and Rachel Ostler *Owner /Agent*
40 Wolfes Road *Address*
Neika 7054 *Suburb/postcode*

Qualified person details:

Qualified person: Kris Taylor

Address: 445 Macquarie Street
Hobart 7004

Phone No: 0476 595 889

Fax No:

Licence No: NA

Email address: office@envirotechtas.com.au

Qualifications and Insurance details: Bachelor of Science with Honours in Geology. Lloyd's Underwriters: soil and rock mechanics, soil and rock testing

(description from Column 3 of the Director's Determination - Certificates by Qualified Persons for Assessable Items

Speciality area of expertise: Geo-technical Reports

(description from Column 4 of the Director's Determination - Certificates by Qualified Persons for Assessable Items)

Details of work: Geotechnical Site Investigation

Address: 40 Wolfes Road
Neika 7054

Lot No: 3

Certificate of title No: 161408/3

The assessable item related to this certificate: Geotechnical Site Investigation written in accordance with AS1726 by a geotechncial practitioner with appropriate experience, training and qualifications.*

(description of the assessable item being certified)
Assessable item includes –
- *a material;*
- *a design*
- *a form of construction*
- *a document*
- *testing of a component, building system or plumbing system*
- *an inspection, or assessment, performed*

Certificate details:

Certificate type: Geotechnical including landslide risk assessment in accordance with “Practice Note Guidelines for Landslide Risk Management 2007” published by the Australian Geomechanics Society.*

(description from Column 1 of Schedule 1 of the Director's Determination - Certificates by Qualified Persons for Assessable Items n)

This certificate is in relation to the above assessable items, at any stage, as part of – *(tick one)*

● building work, plumbing work or plumbing installation or demolition work

OR

○ a building, temporary structure or plumbing installation

In issuing this certificate the following matters are relevant –

Documents: Enviro-Tech Consultants Pty. Ltd. 2026. Geotechnical Site Investigation for a Proposed Extension, 40 Wolfes Road - Neika. Unpublished report for Hamish and Rachel Ostler by Enviro-Tech Consultants Pty. Ltd., 23/03/2026.

Relevant calculations:

References: - AS1726-2017 Geotechnical Site Investigations

Substance of Certificate: *(what it is that is being certified)*

- An assessment of:
- Foundations for proposed building structures.*

Scope and/or Limitations

The Geotechnical Site Investigation applies to the Site and Project Area as inspected and does not account for future alteration to foundation conditions as a result of earth works, drainage condition changes or variations in site maintenance which are not included within the provided plans.

***This report contains soil classification information prepared in accordance with AS2870 as well as AS2870 extracts which may be used as general guidance for plumbing design. The hydraulic designer is to use their own judgment in the application of this information and this report must be read in in conjunction with hydraulic plans for the proposed development.**

I certify the matters described in this certificate.

	Signed:	*Certificate No:*	*Date:*
Qualified person:			23/03/2026